Attachment E

Feasibility Design and Estimate

Attachment E to the ALP Project Draft Supplemental Environmental Impact Statement (DSEIS) includes a description of the design assumptions and a summary of the design calculations, drawings, and cost estimates of the principal structural features of the ALP Project Preferred Alternative. Also attached is a discussion of potential cost allocation for the structural components of the Preferred Alternative.

ANIMAS-LA PLATA PROJECT
FEASIBILITY DESIGN AND ESTIMATE

1.0 PROJECT PLAN AND STRUCTURAL FEATURES

As discussed in Chapter 5 of the Animas La-Plata Project (ALP Project) Draft Supplemental Environmental Impact Statement (DSEIS), following evaluation of the ALP Project alternatives, Refined Alternative 4 was selected as the Preferred Alternative. This attachment summarizes design assumptions and calculations, provides drawings, and presents cost estimates for the structural components of the Preferred Alternative. The design and cost estimates have been prepared at the feasibility level. As such, they provide greater detail than an appraisal-level analysis, but less detail than that required for final construction drawings and design specifications.

1.1 Project Plan

The Preferred Alternative would provide a water supply of 112,000 acre-feet/year (afy) to implement the Colorado Ute Water Rights Settlement Act and serve additional municipal and industrial (M&I) water needs in the project area of southwest Colorado and northwest New Mexico. The Preferred Alternative would regulate project water supply with the off-stream, 120,000 acre-feet (af) capacity, Ridges Basin Reservoir. The reservoir would be filled with water pumped into Ridges Basin from the Animas River, two miles south of the center of Durango, Colorado. From this Durango Pumping Plant, water would flow through a 2.2-mile conduit to the reservoir formed behind Ridges Basin Dam that would be constructed across Basin Creek. The 120,000 af capacity of Ridges Basin Reservoir would include 30,000 af of recreational or fishery storage which would be maintained as a minimum inactive pool. The balance of 90,000 af would be active storage. The reservoir would be operated to regulate the pumped inflow to meet M&I water supply demands and comply with the requirements of the San Juan River Basin Recovery Implementation Plan (SJRBRIP). Water would be available for distribution from the reservoir by connection to the outlet works at the base of the dam, by discharge into Basin Creek where it would flow into the Animas River for downstream use, and by direct pumping from the reservoir to higher elevations.

1.2 Structural Features

Major structural features of the Preferred Alternative are the Ridges Basin Dam and Reservoir, an intake structure and pumping plant of 280 cubic feet per second (cfs) capacity on the Animas River below Santa Rita Park (formerly Gateway Park) in Durango, and an inlet conduit to the reservoir from the pumping plant along a route south of Bodo Creek and County Road 211. The feasibility design for Ridges Basin Dam proposes a zoned earthfill dam containing a thick impervious core bordered by filters and drains and supported by sloping pervious shells upstream and downstream. The height of the dam, streambed to crest would be 217 feet and the length of the crest 1,670 feet. The crest elevation would be 6,892 feet; the full level would be 6,882 feet; and bottom of active storage would be 6,801 feet. Upstream and downstream slopes would be 2:1 (horizontal to vertical) in the active storage range with a bench at the bottom level of active storage and below that level; 3:1 upstream and 2-1/2:1 downstream. The core would bear directly on the foundation rock and the compressible alluvium would be removed both upstream and downstream for placement of the shell of the dam. Foundation excavation would amount to 2,600,000 cubic yards (cy) and dam fill would require 5,700,000 cy.

The Durango Pumping Plant feasibility design includes an intake structure with trash grate, fish screens and fish bypass conduit to the river; a pumping plant building housing five 56 cubic feet per second (cfs) horizontal centrifugal pumps with a total capacity of 280 cfs. The design would also include four smaller pumps to handle lower river flows, to trim between the larger units, and to provide redundancy for reliability; and an electrical switchyard next the building to receive power from the Western Area Power Administration transmission line and supply electricity for the motors and plant. The Ridges Basin Inlet Conduit feasibility design proposes a 66-inch diameter pipeline buried along a route from the pumping plant south across CR 211 and up Bodo Draw south of the creek line, on the alignment described in the 1996 FSFES, terminating at a discharge structure over the crest of the ridge. Water from the discharge structure would flow in a rock-lined channel leading to the reservoir. Hydraulic surge in the conduit would be controlled with a spherical surge chamber located next to the pumping plant, away from the river.

1.3 Prior Designs

The Preferred Alternative represents a large reduction from the facility proposed in the Definite Plan Report issued in September 1979. At that time, the U.S. Bureau of Reclamation (Reclamation) described a project that would provide 80,100 afy of M&I water supply and 118,100 afy of irrigation water. The proposed dam and reservoir capacity was 273,000 af and the pumping plant flow rate was 480 cfs. Comparisons with prior designs in this report relate to the structures described in the Definite Plan Report Appendix A, Designs and Estimates, September 1979, and to the modifications in dam alignment and pumping plant configuration described in the Final Supplement to the Final Environmental Statement (FSFES) of 1996.

1.4 Relocation of Interferences

1.4.1 Gas Pipelines

Four gas pipelines lie within the reservoir area. The three owned by Northwest Pipeline Corporation (Northwest) and Mid-American Pipeline Company (MAPCO) would have to be relocated to permit dam construction to proceed. A relocation route analysis was prepared and the preferred relocation corridor is south of Ridges Basin on portions of Southern Ute Indian Tribal land. Reclamation is working with the Southern Ute Indian Tribe to identify and address their concerns. A line owned by Greeley Gas Company lies along County Road (CR) 211 and would be relocated up the slope near the CR 211 relocation (discussed below) and the Tri-State Generation and Transmission Association electric line.

1.4.2 County Road 211

Portions of the existing CR 211 would be inundated by the reservoir and would be relocated above the reservoir's high water level. Two routes have been investigated. Each would begin at CR 211 on the west side of the crest of Bodo Draw and proceed west about 1.3 miles along the low hills north of the proposed reservoir and near the 115-kilovolt (kV) Tri-State Generation and Transmission Association transmission line. One alternative would be to relocate a portion of the road up a draw and continue westerly on top of the ridge 1.8 miles to an intersection with Wildcat Canyon Road (State Highway 141) at the entrance to the Rafter J residential area. The other alternative would be to continue west, cross the electric transmission line and continue 1.2 miles on the uphill (north) side of the transmission line to junction with existing CR 211 west of the future high water level.

1.4.3 Electrical Transmission Line

A 0.6-mile portion of the 115-kilovolt Tri-State Generation and Transmission Association transmission line would be affected by the planned full reservoir water surface at 6,882 feet elevation. Six structures that would stand in up to 12 feet of water at their present location would be relocated westerly to higher ground.

1.5 Access Roads

Access for project construction activities would be from CR 211, a dirt road. It branches from South Camino del Rio close to a signaled intersection with State Highway 160/550 in Durango. Space for construction equipment and supplies and for worker parking would be available in the reservoir basin for the dam and to the south of the building site for the pumping plant and inlet conduit.

Future access for operation and maintenance of the dam would connect with CR 213, La Posta Road, and proceed along the general alignment of existing private roads to the materials extraction site and haul road that was used for the Uranium Mill Tailings Remedial Action Program in 1990. This site has been designated Borrow Area B and would be expanded as a select materials source for Ridges Basin Dam. Between CR 213 and Borrow Area B, improvement or construction of 1.6 to 2.0 miles of road would be required, depending on the route agreed upon, and Borrow Area B to the dam would require 0.8 miles of improvement and 0.6 miles of new construction. A roadway across the downstream slope of the dam and 0.5 miles of new road on the right abutment would provide access to the dam crest.

1.6 Land Acquisition

Reclamation currently owns 4,638 acres of land in the Ridges Basin area. For project construction, proposed acquisitions include about 800 acres to complete the reservoir land, about 830 acres for the borrow area and access, 46 acres for the pumping plant, and easements for increased flows in Basin Creek and for improvement and future use of access roads from CR 213.

1.7 Construction Program

Project construction would span a period of five to five and one-half years. Beginning with final design engineering, the relocation of gas pipelines would start while the specifications and construction documents are being completed for the dam. At the dam site, excavation of the tunnel portals and tunnel construction would start once the gas lines are removed, about 6 to 8 months from project start. While tunnel construction is underway, the cut-off wall and dewatering wells would be installed, the outlet works stilling basin and channel constructed with the objective of completing stream diversion into the tunnel within 18 months under the dam contract or about 24 to 26 months along the project schedule. The pumping plant and conduit work would begin with equipment delivery times on the order of 12 to 14 months anticipated. Foundation excavation at the dam may be programmed for 8 to 10 months and embankment construction for 20 to 30 months depending on whether double shifts are used.

1.8 Project Operation

Pumping plant and dam outlet works operations would be controlled from the control room of the Durango Pumping Plant. The control room would be in communication with the Reclamation office in Durango where operation of southwestern Colorado projects is coordinated. River flow, reservoir level, outlet flows and upstream watershed gage data indicative of changes in river flow, would be directed to an operational model to advise of the best combination of pumping units to meet the reservoir and downstream demands and comply with the river bypass requirements and downstream commitments. Equipment maintenance duties and inspection patrols of the dam and reservoir would be directed from the pumping plant. Equipment and facility repair tasks beyond the scope of periodic maintenance duties would be assigned to specialized contractors.

2.0 RIDGES BASIN DAM AND RESERVOIR

2.1 Regional Geology And Seismicity

The proposed Ridges Basin dam site is located near the eastern margin of the Colorado Plateau in the Navajo physiographic section. To the north of the site area, the high peaks of the Needle, La Plata, and San Juan Mountains form the picturesque Southern Rockies physiographic province. The Upper Cretaceous sedimentary rocks that underlie the site area form part of the northern margin of San Juan River Basin, a structural basin approximately 100 miles in diameter. The bedrock near the dam site consists primarily of Upper Cretaceous age (70,000,000 years ago) sandstone, shale, siltstone, and coal. The formational names assigned to these upper cretaceous sediments, in order of oldest to youngest include: Point Lookout Sandstone, Menefee Formations, Cliff House Sandstone, Lewis Shale, Pictured Cliffs Sandstone, Fruitland Formation, and the Kirkland Shale.

The major structural features of the region were formed millions of years ago during the late Cretaceous to early Tertiary Laramide orogeny (Kilgore, 1955; Kelly, 1957; Ridgley, et al., 1978). The Upper Cretaceous rocks of the dam site area form a hogback monocline as part of the northern boundary of the San Juan Basin. These rocks are inclined 8 to 32 degrees towards the southeast into the basin and gradually flatten until they are nearly horizontal in the center of the basin fifty miles to the southeast. Much younger surficial deposits in the dam site area include alluvium, alluvial fans, colluvium, landslides, and glacial outwash deposits. The glacial outwash deposits are in the vicinity of Borrow Area B, about 2 miles southeast of the dam site.

A detailed discussion of seismicity of the region is contained in Reclamation's Seismotectonic Report 92-2 (Reclamation 1992). The report concluded that the potential seismic source closest to Ridges Basin dam site is a random (floating) earthquake, unrelated to any mapped faults. This random source is assigned a maximum credible, local magnitude of $6.5M_L$ and would include any surface-rupturing earthquakes on faults within the vicinity of the dam. Using historic seismicity data for the region, a probabilistic analysis resulted in an annual probability of occurrence of $2x10^{-5}$ for a $6.5M_L$ earthquake about 14 kilometers from the site. Other potential seismic sources were considered; however, none of these exceeded the assigned magnitude for the random earthquake or were closer than 14 kilometers from the site.

2.2 Site Geology and Seismicity

The principal bedrock units exposed at the dam site include the Lewis Shale, Pictured Cliffs Sandstone, and the Fruitland Formation in ascending order. The Lewis Shale consists predominately of siltstone of deep-water marine origin. Siltstone logged by the Reclamation (1992) in drill holes is typically calcareous, nonfissile, well indurated, dark gray to black, laminated to very thinly bedded with predominately convoluted bedding, and moderately soft. In surface outcrops, the Lewis Shale weathers to rusty, olive gray platy pieces surrounded by silty, clayey soils.

The Pictured Cliffs Sandstone consists of a massive sandstone (approximately 90 feet thick) ledge underlain by interbedded sandstone and siltstone. The massive sandstone unit forms predominant cliffs at the dam site, particularly on the left abutment. This sandstone unit is typically fine-grained, quartzose, moderately cemented, light gray, laminated to massively bedded and hard, with interbedded sandstone having similar properties. In surface outcrops, the Pictured Cliffs Sandstone weathers to nonfissile, well indurated, dark gray to yellowish brown, moderately hard to moderately soft, blocky ledges.

The Fruitland Formation consists of a non-marine sequence of interbedded sandstone, siltstone, shale and coal. The sandstone is typically fine-grained, quartzose, light gray, laminated to thickly bedded and hard. The siltstone and shale are typically carbonaceous, occasionally sandy, dark gray, laminated to thinly bedded with convoluted bedding common, and moderately hard to moderately soft. The coal is black, brittle, laminated to thinly bedded, soft and generally moderately fractured.

Within the proposed dam embankment footprint, the surficial deposits consist of colluvial veneers on both abutments, alluvial fan deposits that interfinger with predominantly fine-grained fluvial and alluvial deposits of clay and clayey silt. The clay deposits are up to 90 feet thick in the central portion of the dam. Further upstream within the reservoir area, these clay deposits have been designated by the Reclamation (1992) as "Borrow Area A." Downstream, about 2 miles to the southeast, a fairly large deposit of glacial outwash materials consisting of cobbles, gravels, sand, silt and some minor clay exists. These coarse grained deposits have been designated as "Borrow Area B" by the Reclamation (1992).

2.2.1 Abutments

The right abutment is covered by up to 40 feet of colluvium interfingering into alluvium at the base of the slope where the maximum thickness occurs. Bedrock to be encountered consists of sandstone and interbedded sandstone and siltstone units of the Pictured Cliffs Sandstone and siltstone with sandstone interbeds of the Lewis Shale. The right abutment of the planned embankment does not encounter the coal-bearing Fruitland Formation. Depth to groundwater ranges from about 220 feet near the dam crest to about 45 feet near its base.

Sandstones within the right abutment will require only minimal surface stripping of loose blocks and intensely to moderately weathered areas to obtain suitable foundation. Some minimal drilling and blasting or dental concrete will be required in areas of overhangs and ledges. The siltstones and shales have been identified as being moisture-sensitive and susceptible to slaking upon exposure. Prolonged exposure may require secondary excavation and cleanup.

The left abutment is covered by approximately up to 10 feet of colluvium in the higher elevations with up to 15 feet of colluvium and alluvial fan deposits near the base of the slope. Bedrock consists of sandstone, siltstone and interbedded sandstone and siltstone units of the Pictured Cliffs Sandstone and siltstone with sandstone interbeds of the Lewis Shale. The Lewis Shale will be encountered only during excavation at the base of the left abutment. Groundwater will be encountered during excavation at and near the base of the left abutment and will require dewatering.

The blocky sandstone overhangs on the left abutment will require line drilling and blasting with the use of a hoe-ram in other areas for shaping. Some of the sandstone units are very hard and will require close space drilling and blasting. The Lewis Shale can be excavated by common methods and may be susceptible to air slaking requiring secondary cleanup.

2.2.2 Valley Foundation

The valley alluvium extends up to 90 feet thick and consists of sandy clay, clayey sand and lean clay with varying amounts of gravel. Gravel lenses are encountered more along the margins of valley as the alluvial fans interfinger with the finer-grained deposits. The boring logs also show the presence of more gravel beneath the upstream shell of the dam (Reclamation 1992, 1996). The dry density of the valley deposits range from about 85 to greater than 100 pounds per cubic foot (lbs/ft^3), with the higher densities where more sand and gravel occurs.

Groundwater occurs under unconfined conditions within the fine-grained alluvium and within secondary fractures systems of the bedrock. The groundwater table is 30 to 40 feet below ground surface, except within the deeply eroded creek channel where groundwater is near the surface.

For the 273,000 af dam and reservoir proposed by the Reclamation (1996), the soft, compressible upstream alluvium would be left in place beneath the dam embankment and would be consolidated over a two and one-half year period with a wick-drain/surcharge fill system. With the smaller dam of the 120,000 af reservoir, this approach becomes less cost effective and less technically feasible for the following reasons:

1. Permeability of foundation soil (alluvium) is one of the important factors that may affect the consolidation times or effectiveness of the wick drains. This parameter is assumed to be 10^{-6} cm/sec for both horizontal and vertical directions. This assumption may not be true, because horizontal permeability is always higher than the vertical permeability. As Reclamation points out in their analyses, the consolidation time will be affected by the alluvium permeability. Higher soil permeability tends to prolong the consolidation time (refer to Reclamation technical memorandum No RB-3620-22, Table 2, 1996). The laboratory consolidation tests on samples taken from foundation soils indicate that upstream alluvium has a higher Cv (higher permeability) than downstream or central zones. Sandy and gravelly lenses are suspected to be present in the upstream alluvium. If this is true, the use of a coefficient of permeability of 10^{-6} cm/sec for both horizontal and vertical directions for upstream alluvium may not be representative of field conditions. Consequently, the design calculation may underestimate the consolidation time from 3 years to 9 years if the in-situ permeability is changed from 10^{-6} to 10^{-5} cm/sec. In addition, in-situ soil permeability is best obtained from a field pumping test, rather than extrapolating from non-representative laboratory consolidation tests.

2. The anticipated loading conditions will extend beyond the limits of previous projects that have used wick drains to accelerate consolidation. The height of embankment which would be placed above the wick drains is considerably greater than typical applications, and the Consultant Review Board members recommended the maximum surcharge embankment height be limited to 125 feet. With this limitation, only the upper one-third of the embankment foundation could be surcharged.

3. Even with the 125-foot surcharge height limit, the possibility of buckling and lateral shear of the wick drains still exists. The flexible wick drain may potentially buckle under high surcharge load in the order of 125 feet as stipulated by the Consultant Review Board. In addition to high axial load, the flexible wick will also be subjected to high lateral loads induced by the embankment. This could potentially tear the wick drains and develop a (weak) failure plane in the foundation soil.

4. Even if lower permeabilities are present, the two and one-half year period to achieve 90% consolidation for the upper one-third of embankment foundation is not a practical consideration, when this material could simply be removed.

5. The excess pore pressures expected from the wick drain/surcharge system would require costly instrumentation and monitoring of the foundation soils and the embankment during construction and for a period after construction. The concept of staged construction method to strengthen the alluvium foundation is good. It would eliminate the potential settlement of foundation soil and increase the alluvium soil strength. However, this construction method may need an expensive geotechnical instrumentation, monitoring program and engineering evaluation to verify if the increased soil strengths meet the design criteria. Potential changes in the construction schedule may occur if the expected field performances do not meet the design criteria.

6. Installation of wick drain may encounter refusal due to potential presence of stiff clay and dense sand or gravel. The presence of more gravel and dense clay within the upstream valley alluvium would limit the installation of wick drains without a lot of pre-drilling.

For these reasons, the proposed design requires removal of the alluvial materials beneath the dam embankment both downstream upstream. The planned excavation below the water table requires a soil-bentonite cutoff wall just upstream of the upstream toe of the dam embankment and downstream of the excavation for the approach channel. This groundwater barrier would be from 40 to 95 feet deep and extend across the valley floor about 1,750 feet. A series of dewatering wells upstream of the soil-bentonite cutoff wall is also planned. Dewatering trenches and sumps would be required throughout the valley floor excavation in bedrock.

Most of the bedrock to be exposed by the valley floor excavation is Lewis Shale with lesser amounts of Pictured Cliffs Sandstone in the downstream left and right sides of the channel. Excessively weathered and soft areas of the Lewis Shale would require over-excavation and backfill with lean concrete or compacted materials.

2.2.3 *Faults, Joints, Fractures, and Other Bedrock Defects*

No significant evidence of faulting was encountered in any of the Reclamation (1992) exploration drill holes or geologic mapping at the site. One insignificant, bedrock fault was observed in a road cut about 580 feet downstream of the toe of the dam. No other evidence of faulting, such as surface offsets or a break in correlation of lithology has been found. If a fault does exist, other evidence surrounding the site suggests it would be at least of Tertiary age and would not have any impact on dam design.

Sedimentary bedding at the dam site ranges from approximately 20 to 22 degrees towards the southeast near the top of the left abutment to 28 to 35 degrees toward the southeast on the right abutment. This steeping of dip is thought to reflect a point of flexure on the monoclonal ridge that forms Basin Mountain.

Left abutment bedding joints strike N32° to 57°E with dips ranging from 12° to 32°SE. Joint spacing ranges from a minimum of one-quarter inch near the surface to about 7 feet at depth. Most joints are slightly open to tight and often have clay or other material coating and infilling. The joints are moderately rough surfaced.

On the right abutment, bedding joints are oriented N49°E and dip 27°SE. These joints are spaced from 1/32 inch to 1 foot apart near the surface to up to 21 feet at depth.

Three predominant joint sets are described by the Reclamation (1992) as "A," "B," and "C" for the two abutments and valley floor bedrock, with the "A" set subdivided into 2 subsets as follows:

Bedding:	N43° to 49°E, 23° to 27°SE
Joint Set A-NE:	N07° to W, 60° to 82°NE
Joint Set A-SW:	N07° to 24°W, 75° to 76°SW
Joint Set B:	N66° to 76°W, 75° to 77°NE
Joint Set C:	N75° to 78°E, 56° to 71°NW

The joint lengths range from a few feet to up to 100 feet long, spaced very close (less than ¼-inch) at the surface to 50 feet at depth. The average joint spacing is 5 to 10 feet. Most have coatings or in-filling of clay, silt, or other materials.

2.2.4 *Potential Seepage*

Seepage through the bedrock is not expected to be excessive, particularly in the Lewis Shale. However, for feasibility level planning, grouting is considered to control seepage within the bedrock. The proposed design includes a single line grout curtain placed 30 feet upstream of the dam centerline. Primary curtain holes would be drilled to a depth of about 95 feet and spaced on 20-foot centers. In some areas, secondary grout holes spaced on 10-foot centers may be needed. Grout holes would be staged down under a grout cap in 30-foot stages. All grout holes would be inclined about 30 degrees from the vertical in northwest direction.

Additional blanket grouting may be required in localized areas of both of the abutments, particularly in the more fractured and closely jointed sandstone units of the Lewis Shale and Pictured Cliffs Sandstone and at the contact between these two units. The right abutment does not encounter the Fruitland Formation, so the more extensive grouting planned for the prior higher dam embankments will not be needed; however, the contact between the Lewis Shale and Pictured Cliffs Sandstone would be grouted.

Grout takes are expected to be relatively low, probably in the range of 0.5 bags per foot of drill hole. Isolated grout takes of up to 2 bags per foot can be expected where open fractures/joints are encountered.

2.2.5 Reservoir Leakage

Reservoir leakage is expected to be minimal because the majority of the reservoir would overlie the Lewis Shale, weathered portions of the Lewis Shale, and the clayey soil deposits formed from weathering of the Lewis Shale. These materials are expected to be relatively impermeable. There is a gas well drill hole located just upstream of the embankment toe. This drill hole would be closed and sealed in accordance with State and Federal guidelines.

2.2.6 Landslide Potential and Rim Stability

Several small, shallow earth flows have occurred on the slopes of the right abutment. These shallow slumps occurred along the soil/bedrock boundary and would be removed or treated during construction.

A large rotational rock slump occurred about 1,000 feet upstream of the dam axis on the left side of reservoir. This failure occurred along the "A" set joint set and moved downslope below the sandstone cliff. Although no age has been determined for this failure, there is no evidence for recent movement. This area would not be impacted by dam construction but would be inundated, in part, during reservoir filling. During the preliminary design stage, further evaluation of the potential landslide impacts during reservoir filling and lowering would be conducted.

2.2.7 Site Seismicity and Design Earthquakes

Seismic activity in the vicinity of Ridges Basin Dam is low. U.S. Geological Survey (USGS) catalogs of active faults for the State of Colorado do not show any active faults within a radius of 176 kilometers of the site. Based on the probabilistic earthquake ground motion map published by the national Earthquake Hazards Reduction Programs (NEHRP) in 1997, the dam site is considered having a low spectral acceleration (Sa) in the order of 0.3 g at a period of 0.2 seconds for a 2,500-year seismic event. This correlates to a peak bedrock acceleration of about 0.2 for a 2,500-year earthquake. The design pseudo-static seismic coefficient is taken at half the peak bedrock acceleration (Hynes, et al., 1987) under long-term conditions (steady state and partial rapid drawdown). Under short term conditions (end of construction), the recommended pseudo-static seismic coefficient is 0.05 g.

Based on the Reclamation (1992) studies, the random event was assigned a maximum credible earthquake (MCE) local magnitude (M_L) of 6.5at a distance of 14 kilometers from the dam site. Using the Toro, et al. (1997) attenuation relationship for earthquakes in the Central United States, the estimated peak bedrock acceleration at the dam site is about 0.3 g with bedrock response spectra of 5% damping. These design parameters are used to conduct the permanent seismic deformation analysis of the embankment during a MCE event.

2.3 Construction Materials and Their Assumed Properties

Borrow Area A upstream of the dam site is the primary source for the impervious clay core. Reclamation subdivided Borrow Area A into four subareas and summarized the average physical properties of the soils within the subareas (Reclamation 1996). To maintain a plasticity index (PI) of 15 or higher, subareas A1 and A2 appear best suited as potential sources. There may be localized pockets of clay within the embankment foundation excavation that would be suitable for the impervious clay core, and as well, the surficial clay deposits in Borrow Area B could be used. Handling, scheduling, haul distances and other factors would govern which sources are used; regardless, the quantity is sufficient to build a wide, impervious clay core embankment with pervious upstream and downstream shells.

For the impervious clay core material, the effective strength of $\text{Ø}' = 30°$ and $C' = 0$ is adopted for the static stability "end of construction" effective stress analysis (Reclamation 1996). In addition, the wet density of 126 pounds per cubic foot (lb/ft^3) was adopted from the Reclamation laboratory results.

Borrow Area B downstream of the dam site is the main source for the up and downstream pervious shells as well as the internal drain, blanket drain, and transition filter zone between the impervious clay, drain, and pervious shells. The upstream and downstream shells would be constructed from pit run material that is a mixture of minus 24-inch boulders, cobbles, gravel, sand, silt, and minor clay with a maximum of silt and clay approximately 10 percent by weight passing the No. 200 sieve (based on the minus 3-inch fraction). Based on previous projects, experience, engineering judgment, and empirical correlations, these materials should yield an effective strength of $\text{Ø}' = 42°$ and $C' = 0$. A wet unit weight of 135 lbs/ft^3 and a saturated unit weight of 140 lb/ft^3 were assumed for the stability analysis.

Borrow Area B can also provide processed (washed and screened) material for the transition filter zone and the internal drain/blanket drain. The gradation of the transition filter zone material and the drain material would follow the current design standards of the Reclamation, U.S. Army Corps of Engineers, and U.S. Soil Conservation Service as outlined in a recent US Committee of Large Dams publication (Kleiner, 1999).

The transition filter zone material would be poorly graded sand (SP) with 100 percent passing the one-half inch screen and 5 percent passing the No. 200 sieve. An inplace or wet density of 120 lbs/ft^3 and a saturated density of 130 lbs/ft^3 are assumed for the transition filter material. Based on engineering judgment and experience, this material probably has an effective strength of $\text{Ø}' = 36°$ and $C' = 0$. The internal drain material and blanket drain would be poorly graded gravel (GP) with 100 percent passing the 2-inch screen and 0-5 percent passing the No. 4 sieve. A maximum dry density for the drain gravel is approximately 110 lbs/ft^3 a wet unit weight of 130 lbs/ft^3 is assumed. An effective strength of $\text{Ø}' = 40°$ and $C' = 0$ is also assumed.

2.4 Dam Embankment Design

Material specified for the embankment design of the 273,000 af dam proposed by Reclamation in 1996, was a mixture of gravel, sand, silt and clay, having minimum 40 percent passing sieve 200. This material is relatively impervious; therefore, high seepage water pressure would not be expected to dissipate during normal drawdown operation.

The upstream slope was designed to be 2:1 from the dam crest (El. 6,973) to the bottom of active storage (El. 6,898). Based on the Reclamation (1996) stability calculations, the computed minimum factors of safety under partial draw-down from top to the bottom of active storage (approximately 66 feet draw down) was about 0.59 to 0.93, assuming the upstream shell did not have any cohesion (C=0). Judging from these results, Reclamation considered the use of C=0 for shell and core material was very conservative. Subsequently, Reclamation increased the soil strength of upstream shell and impervious clay core by adding cohesion factors of C= 1,500 pounds per square foot (lbs/ft^2) for upstream shell and C= 216 lbs/ft^2 for clay core. Based on this assumption, the computed safety factors under partial drawdown was higher than 1 and considered acceptable.

The common practice among geotechnical engineers is to use a drained soil strength (C=0) approach for long-term loading cases, especially if the shell material is predominantly a cohesionless soil. Therefore, the approach to increase the cohesion factors for shell and clay core is not justified as cohesion can not be counted on for long term stability and under conditions of increased pore pressures during rapid draw down. The previously proposed upstream slope 2:1 above the bottom of active storage level may be potentially unstable during partial and rapid drawdown conditions.

Flattening the upstream slope is probably not an effective way to increase the safety factor of the upstream slope. A better solution is to use pervious shell material especially in the active storage zone, so that high seepage pressures would not be developed in the upstream slope during partial drawdown.

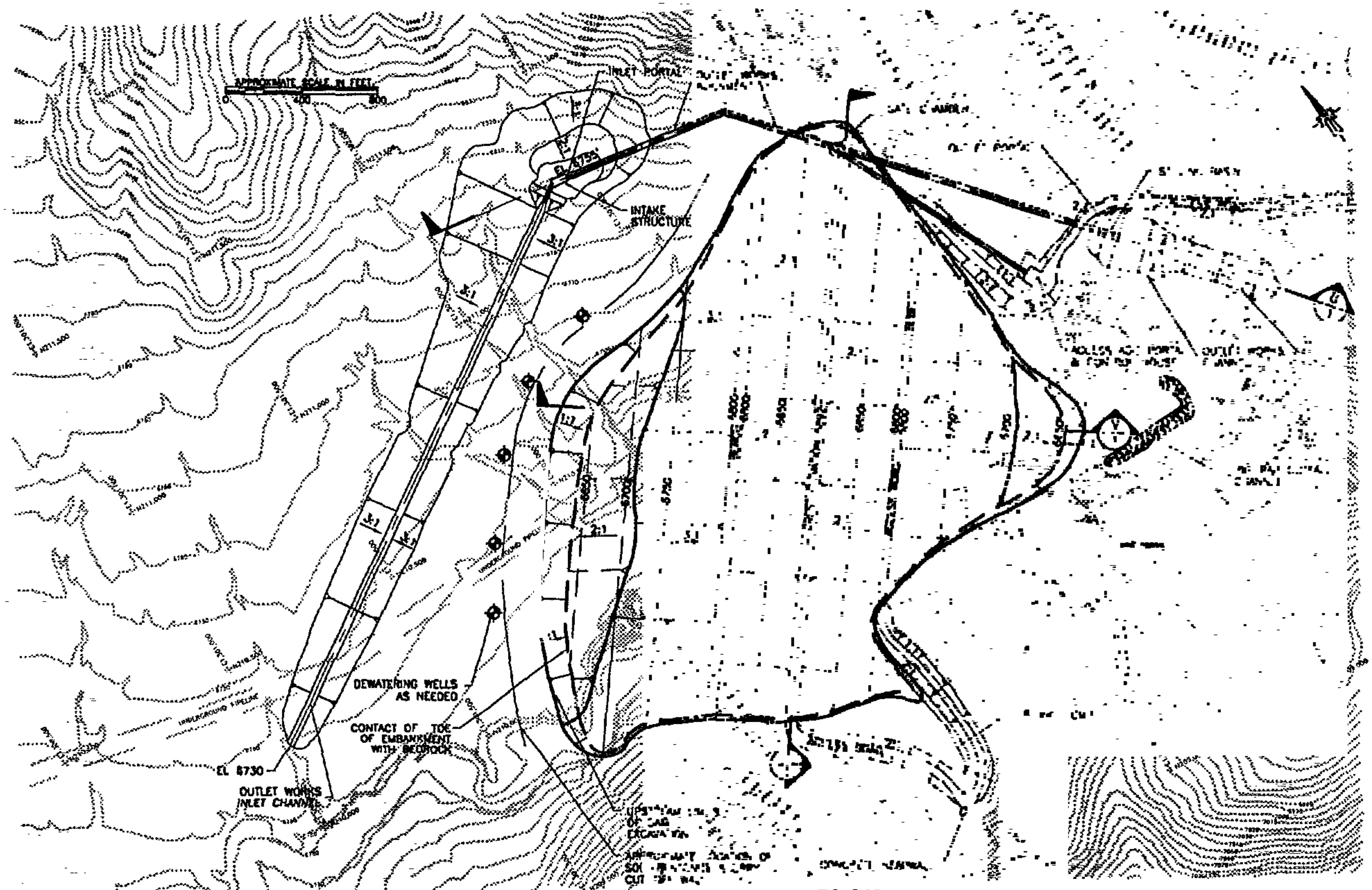
Because of the stability issues with the previous conceptual design and the problems with using the wick drains/surcharge consolidation of the clayey alluvium to be left in place under the upstream portion of the dam, a simpler embankment design is proposed. This preferred embankment design consists of excavating all the soft, loose alluvial materials beneath the entire dam embankment and building a wide, impervious clay core. This clay core would be surrounded by filter transition material and an internal chimney drain connected to a blanket drain on the downstream side. Pervious, boulder/gravel/sand rock fill material would be used for both the upstream and downstream shells. To take advantage of the more than 10 years of Reclamation exploration, laboratory and field testing, and engineering design, the proposed embankment is placed in the valley so the dam centerline is the same as the previous Reclamation alignment for larger dams. Figures 2-1 and 2-2 illustrate the proposed dam in plan view and section.

2.4.1 Foundation Excavation and Treatment

The alluvial materials beneath the planned dam embankment can be excavated by common methods. As noted under the section, Valley Foundation, a soil-bentonite cutoff wall upstream of the upstream toe of embankments is required. This would allow temporary 1 horizontal to 1 vertical (1H:1V) cut slopes in the alluvium. In places, these temporary slopes may have to be laid back at 2H:1V depending on the effectiveness of the dewatering and inplace strength of the alluvium. Within the embankment excavation, dewatering trenches and sumps in the bedrock should be anticipated. As noted earlier, a grout curtain would be placed 30 feet upstream of the dam centerline with primary holes drilled to 95 feet deep, spaced 20 feet apart. The drill holes would be inclined 30 degrees from vertical in a northwest direction to intersect the majority of bedding planes and other rock defects.

2.4.2 Clay Core

The impermeable clay core is planned to be about 420 feet wide at the base of the maximum section and would continue to the crest at 3/4H:1V slopes on both the upstream and downstream sides. The crest width is 30 feet. Ten feet of clay would also cap the transition filter material at the dam crest. The crest elevation (without camber) is planned at elevation (El.) 6,892. During preliminary design, an evaluation of



PLAN

ATTACHMENT E
FEASIBILITY DESIGN AND ESTIMATE

ANIMAS – LA PLATA PROJECT

RIDGES BASIN DAM
PLAN

DRAWN BY: ROC	**REVIEWED BY: DD**	**FIGURE: 2-1**
APPROVED BY: DKR	**DATE: 20DEC99**	

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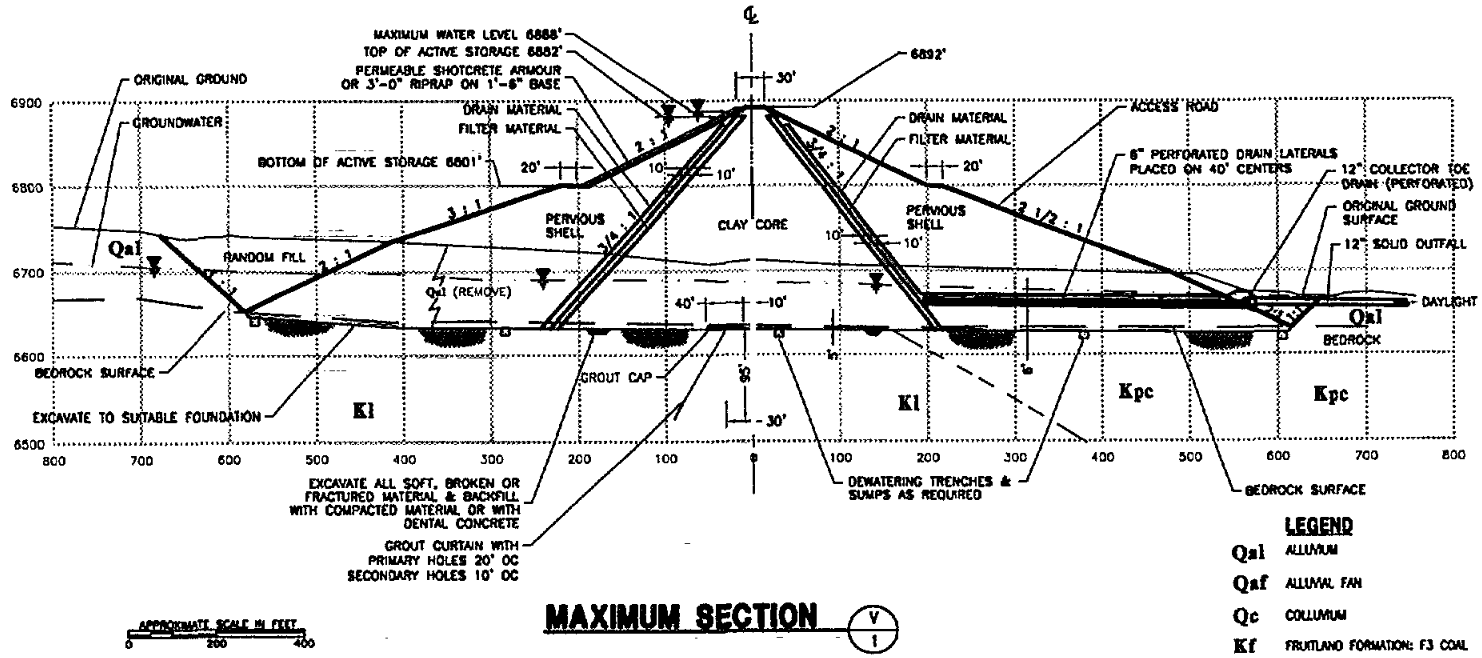


MAXIMUM SECTION V/1

LEGEND

Qal	ALLUVIUM
Qaf	ALLUVIAL FAN
Qc	COLLUVIUM
Kf	FRUITLAND FORMATION: F3 COAL
Kpc	PICTURED CLIFFS SANDSTONE
Kl	LEWIS SHALE

ATTACHMENT E
FEASIBILITY DESIGN AND ESTIMATE

ANIMAS – LA PLATA PROJECT

RIDGES BASIN DAM
MAXIMIUM SECTION

DRAWN BY: ROC	REVIEWED BY: DD	FIGURE: 2-2
APPROVED BY: DKR	DATE: 20DEC99	

the anticipated embankment and foundation settlement would be conducted so that an appropriate camber for the dam can be established. Based on experience and engineering judgment, total settlement at the crest could be as much as 4 to 6 feet. Most of the clay core and upstream shell would be founded on Lewis Shale bedrock where more settlement is expected to occur than embankment founded on the Pictured Cliffs Sandstone.

The final crest elevation of El. 6,892 provides 4 feet of freeboard over the maximum water level from a probable maximum flood (PMF) and 10 feet of freeboard over the top of active storage at El. 6,882.

2.4.3 Transition Filters and Drains

Both the upstream and downstream faces of the impermeable clay core would be covered with 10 feet (horizontal distance) of transition filter material (poorly graded sand) and then another 10 feet (horizontal distance) of drain material (poorly graded gravel). On the downstream side, the chimney drain would connect to a 6-foot thick blanket drain that encases 6-inch diameter, perforated polyvinyl chloride (PVC) drain laterals placed 40 feet on center. These laterals would then connect to a 12-inch diameter, perforated PVC collector toe drain. Three (one in the center and one on each abutment) solid, PVC outfall pipes would then daylight and drain the collector toe drain. Weirs would be constructed for the drain outfall discharge to facilitate flow measurements. In order to daylight the outfalls within a reasonable downstream distance, the blanket drain and laterals with a minimum 3 percent grade are placed about 5 feet below the existing water table and roughly 25 to 30 feet above the foundation bedrock.

2.4.4 Downstream Pervious Shell

From the crest down, the downstream pervious shell has a slope of 2H:1V to El. 6,801 where a 20-foot wide bench and access road is planned. Below this bench the slope flattens to 2.5H:1. Random fill is placed within the toe excavation to restore the original ground surface elevation. As the blanket drain is 5 feet below the existing water table and 25 to 30 feet above bedrock, the pervious shell below the blanket drain would be saturated.

2.4.5 Upstream Pervious Shell

From the crest down, the upstream pervious shell has a slope of 2H:1V to El. 6,801, the bottom of active storage, where a 20-foot wide bench is planned. This 2H:1V slope would be armored with 3 feet of rip rap on an 18-inch thick base or alternatively, shotcrete armor with back drains and weep holes. Below the bottom of active storage, the slope flattens to 3:1 to about where the original ground surface occurs, near El. 6,750. Below this, the temporary pervious fill slope would be 2H:1V to bedrock. The void at the toe of the dam would be backfilled with random material to the original ground surface.

2.4.6 Embankment Quantities

The inplace embankment quantities are estimated to be as follows:

<u>Raw Cut</u>	
Alluvium Excavation	2,323,250 cy
Bedrock Excavation (assumed 5-foot depth)	<u>248,340</u>
Total	2,571,590

A 1.2 bulking factor can be expected for the alluvium and the bedrock is assumed to bulk 1.4 times.

Raw Fill	
Impervious Clay Core	1,245,430
Transition Filter Material	253,010
Chimney and Blanket Drain Material	360,840
Pervious Shells	3,524,160
Toe Fills (Random)	286,900
Total	5,670,340

2.4.7 Stability Analysis

General Assumption and Conditions

Slope stability analyses were conducted for the upstream and downstream embankments using the computer model Stabl5M™. The soil strength parameters were taken mostly from the values that were used in previous Reclamation (1996) slope stability analyses. Based on past experiences and engineering judgment for rock fill dams, a Ø of 42 degrees represents the friction angle of well compacted cohesionless (rock fill) material for the upstream and down stream shells. Separate model runs were conducted for static and earthquake loads (using pseudo-static seismic coefficient of 5 to 10 percent and permanent deformation analyses under MCE event). The analyses were performed for short-term (end-of construction) and long-term (steady state and partial or full rapid draw-down

Condition A : End-of-Construction

Condition A assumes the impervious clay in embankment core and alluvium foundation having undrained shear strength of 3,000 and 2, 000 pounds per square foot (lbs/ft^2). The calculated static factors of safety (FS) are 1.67 and 1.63 for upstream and downstream slopes, respectively. The calculated pseudo-static FS are 1.44 and 1.46 for 5 percent pseudo-static load.

Condition B : High Level Steady State (El. 6,888)

Condition B assumes the impervious clay in embankment core and alluvium foundation having drained friction angle of 28 and 27 degrees, respectively. The maximum water surface in the upstream slope is designed at El. 6,888. The phreatic surface is developed assuming no head loss occurs in the upstream shell and impervious clay core. This would result in higher pore water pressures in the embankments and alluvium foundation; and, hence, is a more conservative approach. Under this condition, the calculated static FS is 2.03 and 1.90 for upstream and downstream slopes, respectively. Under 10 percent horizontal pseudo-static load, the calculated pseudo-static FS are 1.28 and 1.48 for upstream and downstream slopes, respectively. Under the postulated MCE event (magnitude 6.5 with peak ground acceleration of 30 percent), the calculated permanent deformation is very small (less than 2 inches). The design free board of 4 feet against the probable maximum flood (PMF) and MCE event is acceptable.

Condition C: Partial Rapid Drawdown (El. 6,882 to 6,801)

During partial rapid draw down, condition C assumes the pheratic line having a partial head loss occurs in the pervious shell and no head loss occurs in the impervious clayey core. The effective and total stress analyses are used to model the soil strength of the impervious clay core under rapid draw down loading. The effective soil strength was used to model the pervious upstream shell material. The calculated static

FS for upstream slope ranges from 1.5 to 1.65 using effective and total stress analyses. The calculated pseudo-static FS is 1.16 for 10 percent pseudo static load. The computed permanent deformation of the upstream slope under MCE loading is about 3 inches. Since the pheratic surface in downstream slope is similar to steady state loading, the calculated static and pseudo-static FS for both conditions are identical.

Under very extreme conditions, full rapid draw down is conducted from El. 6,882 to 6,701. Assuming partial head loss occurs in the pervious shell and no head loss occur in the clay core, the calculated static FS is about 1.49. Under this remote case, effect of seismic force is always ignored.

In conclusion, the proposed embankment slopes satisfy the minimum factor of safety during end-of construction (FS = 1.3); steady state and partial draw down (FS=1.5); pseudo-static seismic loading (FS=1.1) and full rapid draw down (FS=1.1) assuming the pervious cohesionless material is used in the upstream slope. During partial rapid draw down, the stability of upper slope (above El. 6,801) is primarily controlled by the position of pheratic surface rather than slope geometry. Hence the use of pervious cohesionless material is necessary to induce a significant head loss in the upstream slope.

2.5 Outlet Works

In the conceptual, appraisal level stage of design, a pressurized 78-inch diameter conduit outlet with a capacity of 1,530 cubic feet per second (cfs) was considered to be benched into the left abutment for the 135,000 af reservoir and dam. Even after attempting several alignments, the bedrock cuts through the low knoll on the left abutment would be as high as 90 feet and there would be a short segment where bedrock would require lean concrete fill to maintain a suitable elevation and grade. Bending the conduit did not resolve these short-comings and could create adverse hydraulic conditions. Reclamation (1996) also rejected a conduit outlet for much of the same reasons as follows:

- ❑ The thick alluvial layer in the valley requires the conduit to be placed close to the abutment to maintain the desired grade; and

- ❑ The dam footprint is similar in shape to an inverted cone with a long upstream toe and a short downstream toe. These combinations make it difficult to develop an alignment suitable for hydraulic considerations (minimum bends).

Reclamation (1992, 1996) considered both abutments suitable for a tunnel outlet, with the right abutment being originally favored because of economics. However, during final design, the right abutment alignment disclosed several problems, including problems with both portal cut excavations and difficulties in locating suitable foundations for both the intake structure and stilling basin. Also, the stilling basin would be located too close to the toe of the dam. For these reasons, the Reclamation tunnel outlet configuration on the left abutment is adopted as the preferred structure and alignment for this feasibility level design and cost estimate.

2.5.1 Summary Description of Outlet Works

Outlet works consist of an intake approach channel, intake structure, upstream tunnel, gate chamber, downstream tunnel, separate access adit, control building, stilling basin, and the discharge channel. A plan and section is presented on Figure 2-1.

The outlet works intake approach channel would begin at Basin Creek with an invert El. 6,730 or about 20 feet above the existing water table. The channel with a 20-foot wide base and 3H:1V side slopes, would extend 1,614 feet across the valley terminating at the intake structure. All of the channel excavation would be in alluvium and common methods can be used. The flatter 3H:1V slopes are to provide stability under submerged conditions.

The intake structure consists of a drop inlet structure with its entrance at El. 6,760 and a centerline El. 6,733.75 to insure the drop inlet structure is founded on bedrock. The drop inlet structure would have a temporary opening at El. 6,730 for diversion, which would be plugged with concrete after diversion. A small diversion inlet pipe with a bulkhead gate located in the inlet channel at El. 6,730 and connected to the drop inlet would eliminate 60 feet of dead storage.

From the inlet structure to the gate chamber, a short 7.5-foot diameter conduit section from the inlet would connect to a 7.5-foot diameter, reinforced concrete, pressurized tunnel about 665 feet long. The tunnel would change direction about mid-point with a 41 degree 38 minute bend.

The gate chamber would be located within the left abutment a few feet upstream of the dam axis. A pair of 4 foot by 6 foot bonneted gates in tandem function as the main guard gate and regulating gate for emergency releases. A combination of a 20-inch pipeline, isolation ball valve, and a 14-inch jet flow gate control releases of up to 100 cfs for the regular flows. A second 30-inch pipeline, isolation ball valve, and a 24-inch jet flow gate would be installed to increase the regular releases up to 250 cfs in the future. Both of the discharge pipes from the jet flow gates would be flanged so that connections to future distribution pipelines could be made easily. The working space within the gate chamber would consist of a domed structure with an inside diameter of about 22 feet. Access to the gate chamber working area from the control building would be provided by a 587 feet long access adit from the downstream left abutment of the dam.

The control building would house controls for the outlet gates and valves, flow meters and recorders for release flows, and dam safety instrumentation monitors. It would also house the propane gas-fueled standby generator. Operational and status data would be transmitted by telephone wire or wireless to the Durango Pumping Plant and valve position commands received. Electrical power would be supplied to the control building with an extension of La Plata Electric Association lines from west of CR213 along the roadway up Basin Creek.

The downstream tunnel would be concrete lined, connecting the gate chamber to the stilling basin. This tunnel section would be flat-bottomed, 8 feet in diameter and about 696 feet long. Flow in the downstream tunnel would be open channel.

The stilling basin would have a length of about 60 feet and would be designed to provide energy dissipation for releases up to 500 cfs. Safe downstream channel capacity and maximum planned releases would be limited to 250 cfs. Emergency releases exceeding the stilling capacity of the basin would be flipped downstream of the structure; however, this would be an extreme operational condition.

The downstream channel conveys releases from the stilling basin to Basin Creek, a drop of about 40 feet. The downstream channel would consist of a length of rip rap lined channel; a headwall and a length of reinforced concrete pipe; an impact energy dissipater; and a short length of rip rap lined channel.

2.5.2 Summary of Outlet Works Geology

Tunnel cover varies from 63 feet at the inlet portal, 118 feet at the gate chamber, and 22 feet at the outlet portal, with a maximum cover of 170 feet. Although the entire tunnel is below the water table, groundwater inflow is not expected to exceed 75 gpm. The intake structure and inlet portal would be founded on Lewis Shale and would be excavated below the water table. Approximately 42 feet of alluvial fan deposits would be excavated to reach bedrock at El. 6,730. About 60 percent of the tunnel length would be excavated in Lewis Shale starting at the inlet portal. Pictured Cliffs Sandstone would be encountered about 200 feet downstream of the gate chamber. Methane gas should be anticipated throughout the tunnel excavation, particularly in the Pictured Cliffs Sandstone. The access adit would encounter about 495 feet of Pictured Cliffs Sandstone and about 109 feet of siltstone and shale beds of the Lewis Shale.

According to Reclamation (1992), the Lewis Shale has the following properties:

Compressive Strengths	8,970-20,600 psi 10,150 psi median
RQD Value	70-90

Likewise, the Pictured Cliffs Sandstone has the following properties:

Compressive Strengths	
P2, P5, P7 and P9 Units	9,560-22,200 psi 11,390 psi median
P1, P3, P4, P6 and PP Units	2,544-9,330 psi 7,696 psi median
RQD Value	
P2, P5, P7 and P9 Units	71-100 94 Average
P1, P3, P4, P6 and PP Units	53-100 90 Average

In general, the rock quality designation (RQD) suggests none to occasional steel sets and occasional split sets with wire mesh and steel straps. Shotcrete would not be used extensively because of the wet conditions and bonding problems with the weaker sedimentary bedrock units. Tunneling advance could be 3 to 10 feet, commencing support after each blast and using complete support 50 feet back from the face.

2.6 Basin Creek Improvements

Planned water supply releases from Ridges Basin Reservoir ranging from 25 to 130 cfs and future releases up to 250 cfs are projected for non-binding Colorado Ute Tribe water use development. These releases would be controlled by valves at the outlet works of the dam and would flow down Basin Creek to the Animas River. The watercourse along the creek is about 3.2 miles in length with an elevation drop of about 420 feet from the dam to the river. The upper 2.5 miles of the creek is incised into a clayey sand formation while the lower 0.7 miles passes over several natural rock controls. Planned releases are greater than the normal seasonal runoff and silt transport into the Animas River is a concern. Field survey of the creek channel indicates that a means of control would be necessary if the planned releases are not to increase silt transport. Alternative means of control investigated included the following:

- Armor the channel with rock
- Replace the streambed with a concrete lined channel
- Install a number of check or vortex weirs
- Release flows into a conduit laid along side the creek

The steep slope of the streambed and the absence of a nearby source of heavy, durable rock makes possible use of channel armoring for erosion control less attractive than a concrete lined channel. With the slope of

0.026, velocity in a concrete lined channel would be high and protective measures would be necessary for safety of wildlife and persons. Creating steps in the channel with a series of check and drop or vortex weirs would produce an increase in silt transport initially but would stabilize with use. It would also create some wetlands. The steps would be placed about 150 feet apart. All three of these solutions involves realigning the streambed into gentle curves and grading it to create relatively flat slopes from the modified flow line to the original ground on either side. The lower 0.7 miles of the creek may accept the additional flow without significant modification.

The conduit alternative would require pipe of about 48 inches diameter. It would be laid in roadway and across private property, leaving the creek relatively undisturbed. Use of pressure pipe would leave open the potential for energy recovery at the conduit discharge. The alternative of check or vortex weirs was selected for the cost estimate of this report.

3.0 DURANGO PUMPING PLANT

3.1 Location and Access

The Durango Pumping Plant is the proposed intake pumping plant for the ALP Project. The pumping plant site is two miles south of the center of Durango, on the west side of the Animas River across from the downstream end of Santa Rita Park, formerly Gateway Park. This is the same location proposed in the 1996 FSFES and is presented in Map 2-6 in Chapter 2 of the DSEIS. Access to the site is from CR 211, immediately north of Centennial Mall. A plant entrance road, with 30 feet of gravel surface and drainage ditches would branch from the existing roadway along the base of Smelter Mountain about 400 feet north of its intersection with CR 211, and proceed a distance of 1400 feet at a maximum grade of less than six percent to the pumping plant.

3.2 Pumping Plant Structure

The pumping plant is designed as an indoor structure that houses nine horizontal single stage centrifugal pumps of varying capacities. Oriented with the long side parallel with the river the pump and equipment portion of the plant would be below the finished ground surface with an interior height of 43 feet, a width of 57 feet, and a length of 250 feet. An additional 20 feet of length may be provided for future City of Durango pumping units. Over the below surface portion of the plant the crane housing would extend 24 feet above the ground to facilitate loading, unloading and maintenance of the pumping units and equipment. The crane housing would be 40 feet wide and 250 feet long. Construction would use cast in place and precast concrete. A spherical air chamber would be partially buried behind the plant away from the river. Incoming power lines and an electrical switchyard would be located to the south, between the plant and CR 211. Fill slopes between the plant and the intake structure and between the intake structure and the river leave space for landscaping.

3.2.1 Foundation Conditions

Overburden at the pumping plant site is generally impervious clayey sand about 15 feet thick underlain by eight feet of river terrace deposits. Bedrock at the site is siltstone, sandstone and shale of the Point Lookout Sandstone formation. It occurs at a depth of about 25 feet and would form the foundation for the plant. Groundwater at a fault about 150 feet southeast of the proposed plant may carry slightly elevated levels of contaminants, however, its location is well beyond the planned limits of construction excavation. Backfill around the structure would employ drains or pervious material placed to restore natural groundwater movement after construction.

3.2.2 Construction Dewatering Monitoring

The Durango Pumping Plant site is a former Uranium Mill Tailings Remedial Action site and regular monitoring of the water removed during construction dewatering would be required. The contractor would be required to prepare and implement, if necessary, a contingency plan for treating the water removed during excavation in the event that groundwater contamination levels exceed anticipated limits.

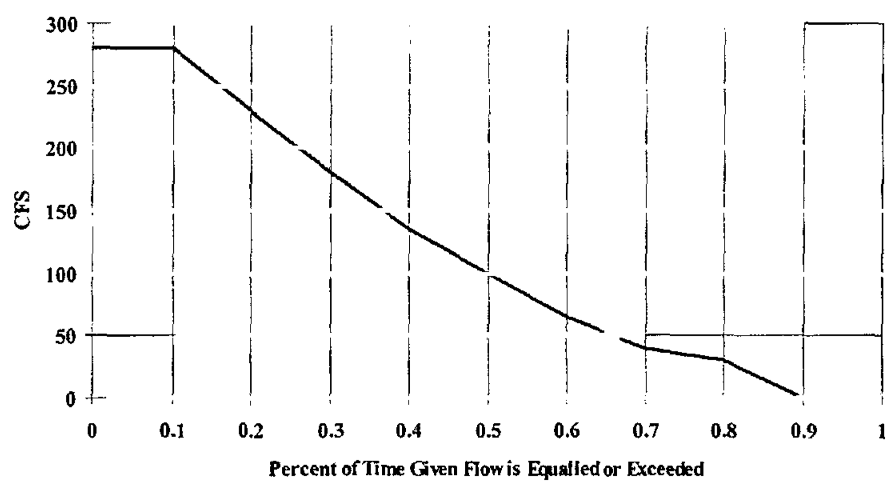
3.3 Intake Structure

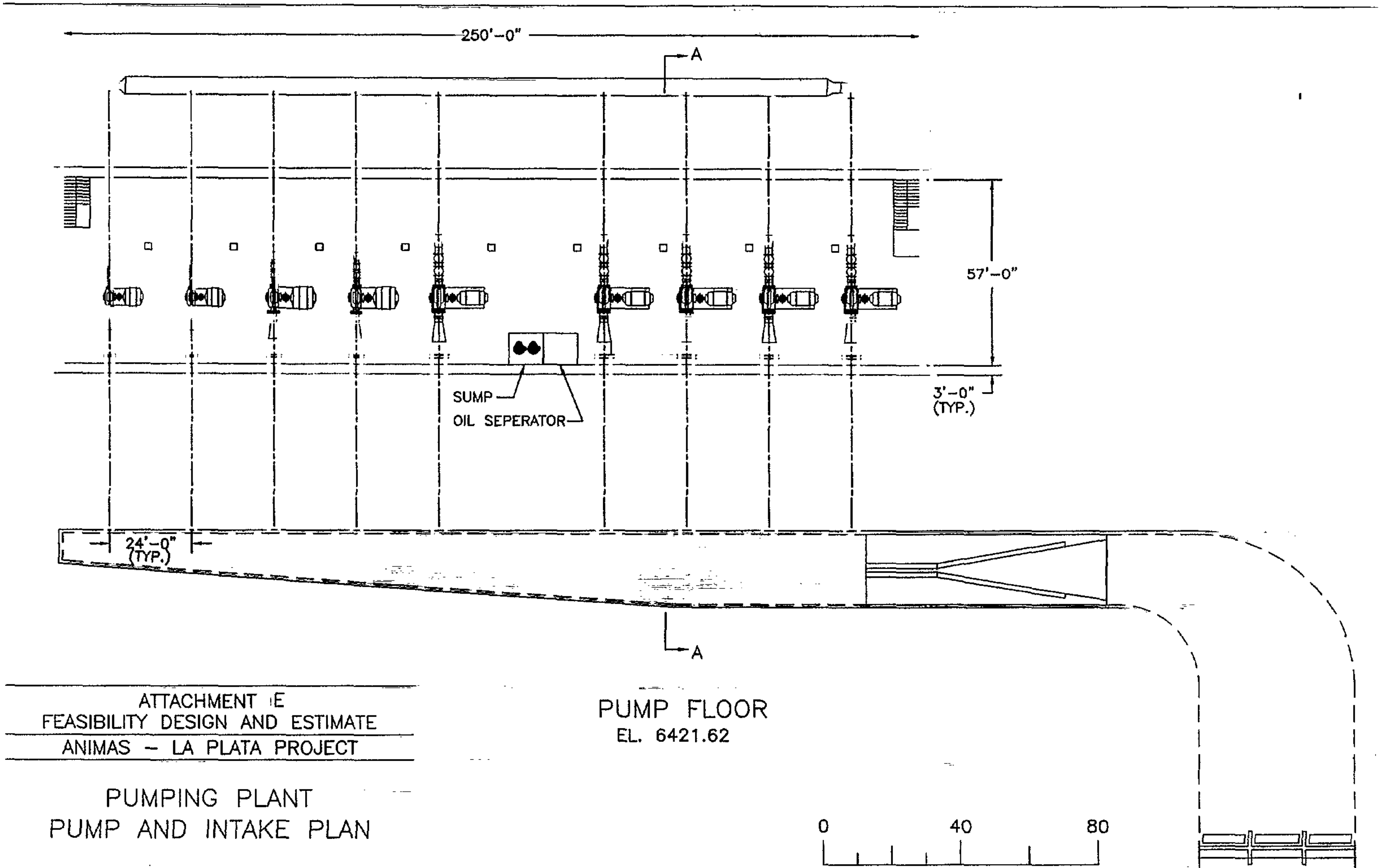
Intake structure feasibility design is based on reducing the flow path dimensions resolved by the model tested in 1996 in proportion to the reduction in design flow rate. Of the two channels in the 1996 design only the longer radius channel is employed in the present design. It consists of a grated structure 48 feet along the riverbank that passes flow through three control gates into a covered channel that extends 90 feet back from the river then turns to pass through a V-type fish screen then enters the covered forebay chamber that supplies the pump intake pipes. Design approach velocity for the screen is less than 0.5 feet per second. The fish screen area is open for cleaning and maintenance access. A fish bypass conduit at the base of the screen designed to carry 15 cfs back to the river would extend downstream about 300 feet to develop one foot of differential head. A plan view of the intake structure and the pump floor is presented in Figure 3-1.

3.4 Pumping Unit Selection

The pumping plant capacity, number of pumps and individual pump capacities were selected to permit the integration of available flow over the duration shown on the probability curve derived from the hydrological model and presented in Exhibit 3-1. Maximum pumping capacity is linked to the reservoir active storage through the hydrological model. For an active storage quantity of 90,000 af the maximum pumping rate was determined to be 280 cfs or 125,700 gpm. Static lift from the river level over Bodo Draw is 511 feet and maximum dynamic losses amount to 40 feet for a maximum total dynamic head of 550 feet. The capacity selection of individual pumps was restricted to single stage, low speed pumps of proved design that were commercially available.

Exhibit 3-1
Inflow Pumping Rate
Probability Curve





ATTACHMENT E
FEASIBILITY DESIGN AND ESTIMATE
ANIMAS – LA PLATA PROJECT

PUMPING PLANT
PUMP AND INTAKE PLAN

DRAWN: D.F.T.	CHECKED: V.P.N.	FIGURE
APPROVED: D.D.	DATE: 9/14/99	3 – 1

Following this criteria and with data from manufacturers it was determined that the maximum flow could be attained with five horizontal centrifugal pumps each rated at approximately 25,000 gpm and 550 ft of total head. Commercially competitive pumps are available in this range. To accommodate lower flows, two intermediate size pumps (approximately one-half of the capacity of the large pumps) and two small pumps (approximately one-half of the capacity of the intermediate pumps) were also selected. These pumps would provide standby capacity should one of the larger pumps be unavailable for service.

3.4.1 Pump Configuration

Large single stage horizontal pumps are similar in silt handling capability to the vertical spiral case pumps proposed for the higher capacity pumping plant of the 1996 ALP roject design and the horizontal configuration is more accessible for maintenance. The horizontal pumps would also require less structure height than the former vertical pumps. The forebay and the pump intake piping arrangement would provide positive suction head at the pumps. These features are presented in Figure 3-2. Operating characteristics of the selected pumps are presented in Table 3-1. For a pumping plant with multiple pumps the superimposition of the pump operating curve and the piping system elevation and friction curve provides a system head curve. This illustrates the flow produced by a single pump or any grouping of pumps. Exhibit 3-2 presents the system head curves for the large pumps.

Table 3-1
Pump Operating Characteristics

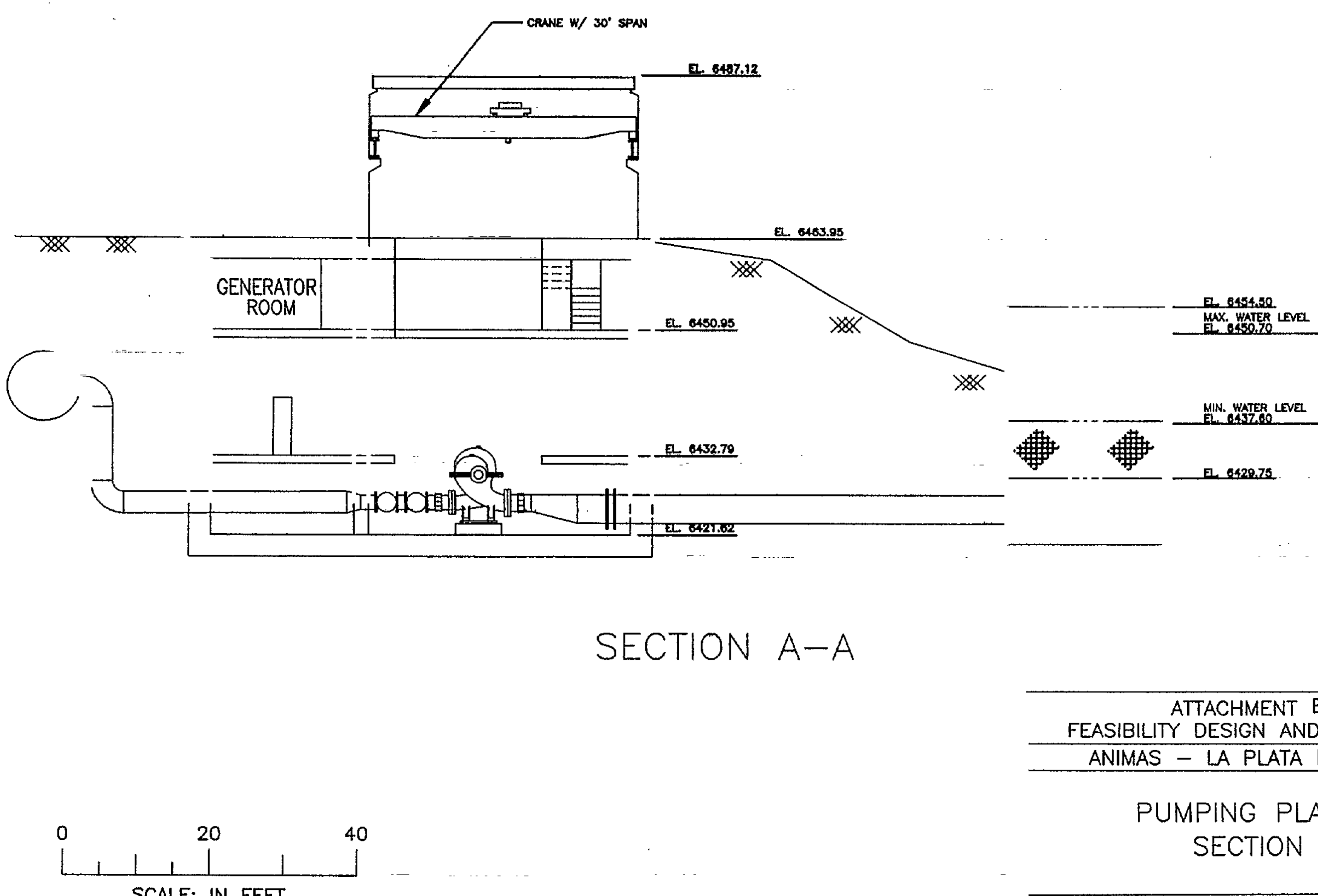
Characteristic	Unit	Pump Size: Large	Intermediate	Small
Flow	cfs	56	28	14
Flow	gpm	25,000	12,500	6,250
Total Dynamic Head	feet	550	550	511
One Pump Run-out Flow	gpm	32,000	15,000	7,000
One Pump Run-out Head	feet	500	500	500
Driver Speed	rpm	900	1200	1800
Driver Power	HP	5000	2500	1.250

3.4.2 Materials and Manufacture

The following materials and manufacture standards would be specified for service conditions at the intake pumping plant:

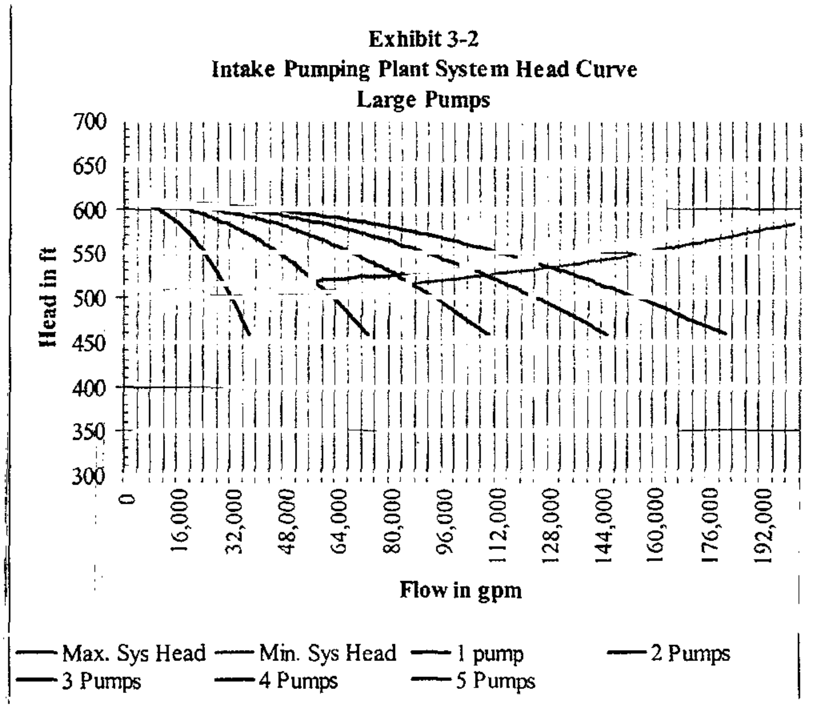
- Casing: Axially split casing with suction and discharge flanges in the lower half. This allows access to the rotating element without disturbing piping or motor drive and also provides rigidity for pipe loads, reducing coupling and bearing misalignment. Upgrade casing to steel rather than cast iron to allow welding should wear result from silt in the water. Upgrade casing wearing ring to 13 percent chrome stainless steel for wear resistance to silt.

- Impeller: Double suction type for hydraulic balance. Upgrade from bronze to 13 percent chrome stainless steel for wear resistance to silt. Provide optional hardened wear rings for better service.



SECTION A–A

ATTACHMENT E FEASIBILITY DESIGN AND ESTIMATE		
ANIMAS – LA PLATA PROJECT		
PUMPING PLANT SECTION		
DRAWN: D.F.T. APPROVED: D.D.	CHECKED: V.P.N. DATE: 9/14/99	FIGURE 3 – 2



- ❑ Shaft and Sleeves: Heat-treated carbon steel and protected from wear and erosion by removable sleeves.

- ❑ Seals: Gland packing type, PTFE impregnated. Seal water, if required would be provided from the pump discharge by way of a filtration system.

- ❑ Bearings: Heavy duty, anti-friction type pump bearings arranged for oil lubrication without external cooling.

3.5 Major Valves

Major flow control and isolating valves would include the following:

- ❑ Pump Discharge Control Valves: Metal to metal seated American Water Works Association (AWWA) Standard Specification C507 Class 250 ball valve. Operated by an air/oil cylinder operator capable of operating the valve with a minimum of 100 psig air pressure to the cylinder. Controls provide for independent opening and closing speed controls as well as an emergency high speed close feature with a separate speed control valve.

- ❑ Pump Discharge Isolating Valves: Metal to metal seated AWWA C507 Class 250 ball valve.

- ❑ Pump Suction Isolation Valves: Rubber seated AWWA Class 25 butterfly valves.

3.6 Plant Auxiliary Systems

The following plant auxiliary systems would be provided to support and operate the pumping units and valves and serve the plant operating and maintenance personnel.

3.6.1 Compressed Air System

Compressed air at a nominal 125 psig pressure would supply the air pneumatic valve operators of the pump discharge control valves, the sewage ejector and utility hose connections throughout the plant for operation of pneumatic tools. The system would include:

- Two station service air compressors (one duty and one stand-by): Two stage, multi-cylinder, automatic unloading, air-cooled, horizontal type, belt driven by an electric motor suitable for operation on 460-V, 3-phase, 60 Hz. Operation normally automatic, controlled by pressure switches mounted on the main air receiver with manual operation available at the motor control center.

- Air receivers: Vertical type with floor supporting stand, inlet and outlet connections, pressure relief valve, pressure gage, automatic condensate trap, drain valve and manhole. Designed for 150 psi working pressure in accordance with Section VIII of the ASME Boiler and Pressure Vessel Code. An air dryer with bypass would be provided on the discharge piping of the main air receiver.

3.6.2 Oil Recovery System

An oil recovery system would guard against pollution from accidental oil spills in the plant. Designed to meet a 10 parts per million (ppm) maximum oil-in-water effluent criterion it would provide a margin of safety over the EPA requirement of 15 ppm stipulated in paragraph 423.12 of the "Environmental Protection Agency Effluent Guidelines and Standards for Steam Electric Power Generating." Oil separators would be installed to recover any oil spilled and collected by the plant drainage system with capacity to contain the largest expected single spill that could occur in the at the hydraulic power units or in the oil storage area. A recovered oil storage tank would store oil for removal for disposal.

3.6.3 Plant Drainage System

The plant drainage system would be designed to collect water from areas where wash down and seepage may collect, from watercooled equipment, and from pump seals and convey it to a sump where it would pass through the oil separator. Any separated oil would go to the recovered oil storage tank and the water would be pumped to discharge into the intake bay.

3.6.4 Service and Domestic Water Systems

The service water system would distribute water to hose connections throughout the plant, supply cooling water for air compressors and aftercoolers and for air conditioning equipment. The domestic water system would supply the restroom facilities and drinking fountains. Both systems would be supplied by a connection with the City of Durango treated water main along the west side of Highway 160/550, about 300 feet east of the pumping plant.

3.6.5 *Sanitary Waste*

Sanitary waste plumbing from the restroom facilities and drinking fountains would drain to the plant sewage ejectors. The sewage ejectors would pump to a connection with the City of Durango sewer system on the west side of Highway 160/550, about 300 feet east of the pumping plant.

3.6.6 *Fire Protection System*

Fire protection for the pumping plant includes:

- Fire suppression sprinkler system in conjunction with portable fire extinguishing and fire hose reel.
- Fire hose stations inside the plant.
- An extension from the City of Durango water main along the west side of Highway 160/550, to provide outside fire hydrants near each end of the plant and near the switchyard.
- Portable 20 lb. wall mounted fire extinguishers, and 100 lb. wheeled units.

In addition, heat detectors would be placed in areas of the pumping station protected by manual extinguishing equipment and ionization detectors would be placed to serve as an early warning system. Fire detection and early warning alarms would be displayed on an enunciator system panel in the control room.

3.6.7 *Stand-by Power*

In event of normal service power loss, diesel generating sets would provide emergency power for essential emergency loads including control room, sump pumps, elevator, limited area lighting, and the unit auxiliaries necessary for safe shutdown of the pumping units.

3.6.8 *Discharge Pipeline Dewater*

Provisions will be included in the design to permit dewatering of the pump discharge, or reservoir inlet conduit. Piping, sleeve valve and isolation valves would bypass the pumps and lead back to the intake works and the river.

3.7 Servicing Equipment

3.7.1 *Crane*

The crane provided over the plant would be a single trolley 20 ton bridge crane designed to comply with OSHA standards and adequate for the weight of the heaviest load to be handled. The crane would be used for handling materials and equipment, including loading, assembly, installation and future maintenance operations.

3.7.2 *Elevator*

A hydraulic elevator would be provided to transport personnel between floors.

3.8 Heating, Ventilation and Air Conditioning

The pumping plant would be provided with heating, ventilating and air conditioning systems to meet the various temperature and ventilating requirements. Continuous air conditioning would be provided for the control room only while all other rooms and spaces would be ventilated by means of forced air systems. The criteria for design would be to use the larger volume computed as (1) the amount of air required to maintain the design temperatures considering heat gains from solar sources, equipment, lights, and personnel and (2) the number of air changes per hour.

3.8.1 Control Room

The control room air conditioning system would maintain a slight positive pressure by adding make-up air equal to the exfiltration losses plus constant exhaust air quantities. In the summer, the make-up would be mixed with return air, filtered, cooled by the cooling coils, and distributed to the conditioned rooms through ducts. A thermostat in the return air duct or room would start and stop the refrigeration compressors to maintain the desired average room temperature. A standby unit would be provided for emergency cooling during maintenance or breakdown of the main unit.

In the winter, the system fan would circulate air to each zone and uncontaminated air would be recirculated. Electric duct heaters provide for heating and for tempering the make-up air.

3.8.2 Exhaust Systems

Exhaust fans would pull air from the plant through the ventilated spaces in the sewage pump area, oil recovery, oil and storage room and diesel generator room and exhaust it directly outside. The oil storage room exhaust duct would be equipped with an automatic fire damper.

3.8.3 Pump and Switchgear Gallery

Heating and ventilating systems for the pumping and switchgear gallery consist of two air handling units and dampers, electric heater, ducts, registers, and control equipment. Each air handling unit would be packaged, including air filters, air plenum and centrifugal type cabinet fan. The ventilation system would include power roof ventilators to exhaust air to atmosphere.

The air handling unit supply fans would run continuously all year and have manual start/stop controls. The power roof ventilators normally cycle on and off as a function of pump room temperature. During summer operation, the system would take in 100 percent outdoor air, supply it to the rooms and exhaust outside. During winter operation, all uncontaminated air would be recirculated and the ventilators would not operate. Heating would be primarily by recirculation of equipment heat within the plant.

3.9 Electrical Features

3.9.1 Pump Motors

Motors for driving the centrifugal type pumps would be horizontal-shaft induction type, five-5,000 horsepower (hp) at 13,200 volts, and two-2,500 hp and two-1,250 hp at 4,000 volts. The enclosure would be specified as Weather Protected Type II with ventilating passages at both intake and discharge to permit passage of external cooling air over and around the windings of the motor. Motor enclosures would also be constructed to limit the motor noise level to 85 decibels or lower. Control and protective devices would be supplied for alarm and shutdown of the motors for problem conditions.

3.9.2 Switchyard

The switchyard to bring power into the pumping plant would be designed and built by the Western Area Power Administration (WAPA). A single transmission line and 13.8 kilovolts (kV) circuit would feed the plant load. Purchasing and storing a spare transformer would provide additional reliability.

3.9.3 Bus and Switchgear

Cables rated at 15 kV would carry power from the utility structure located outside the plant to the 15 kV switchgear located inside the plant. A 15,000-volt bus would run inside the plant for the 13,200- volt motor starters and a 5,000-volt bus for the 4,000-volt motors starters. The 5 kV switchgear, 13,800 to 4,160 V transformer, 4,160 to 480V plant service transformer and 480 volt distribution equipment would all be located inside the plant building.

3.9.4 Motor Starter and Control Equipment

The plant design connected load consists of five-5,000 hp motors, two-3,000 hp motors, two-1,315 hp motors and 500 kilovolt-amperes (kVA) of miscellaneous load. The plant full load current would be 3560 A at 4,160 V. A manufacture search indicated that the largest vacuum circuit breaker available for 5 kV metal clad switchgear is 3,000 A. Since that is not sufficient for the connected load of 3560 A, the larger 15 kV metal-clad switchgear was selected. A 5,000 hp motor is available for 4,160 V, however, the 13,200 V motor was selected to suit the switchgear.

Motor control equipment would contain draw out fuses, starters, control power transformers, selector switches, pushbuttons, and all unit protective and control devices. A programmable motor protective relay would be used to centralize all unit control and protective features.

5 kV Motor Starting and Control Equipment

Cables from the 15 kV metal clad switchgear circuit breaker would provide power to a 7,000 kVA, 3P, 13,800-4,160 V, 60 Hz transformer in the 5 kV switchgear assembly. The secondary side of the 7,000 kVA transformer would be connected to the 5 kV switchgear main circuit breaker using 5 kV cables. A 5-kV auto-transformer with motor control equipment would serve the starting and control functions of the 3,000 hp and 1,315 hp motors.

15 kV Motor Starting Equipment

A 15 kV metal-clad circuit breaker along with a 15-kv auto-transformer would function as the starter for the large 5,000 hp units. Reduced voltage starting would be used to limit the power system voltage drop.

3.9.5 Main Plant Control

Main plant control cubicles and console would serve to centralize all plant operations including indicating, recording, operating, communication, and protective functions. Manual, automatic and supervisory type functions would be provided to allow full flexibility in plant operations. A computer programmable logic controller housed in this control board would provide automatic features such as automatic restart after power failure and selection different size units to optimize plant performance based on available river flow. Control console personal computers would be furnished with the necessary hardware and software to field program each ladder logic diagram.

3.9.6 125 V DC Battery and Battery Charger

A 125 V DC battery and battery charger would be provided to supply control power for 15 KV and 5 kV metal clad switchgear and main plant control cubicles.

3.9.7 Plant Service

Plant service power supply would be obtained by connecting to the 5 kV switchgear fused disconnect switch and providing a 480 V unit substation.

3.9.8 Auxiliary Control Boards and 480-Volt Motor Control Centers

Auxiliary control boards and 480-volt motor control centers would be provided in the plant for operating the auxiliary systems such as hydraulic pumps, water cooling pumps, electrically driven valves, and air compressors. Auxiliary control boards and 480-volt motor control centers would obtain power from a unit substation.

4.0 INLET CONDUIT

4.1 Route and Features

The conduit route from the Animas River to Ridges Basin was selected to provide the lowest pumping lift with reasonable construction access and to minimize alteration of natural terrain contours. Inspection of the reservoir site topography indicates two lower level inlet routes with reasonable proximity to the river. One route would enter the reservoir from Basin Creek and the other would enter the reservoir from Bodo draw. For the design reservoir size of 120,000 af the lift from the river to the elevation of the ridge at the top of Bodo draw is less than the lift from the river to the reservoir at the downstream locations accessible to Basin Creek. Placing the conduit at a lower elevation by excavating the crest of the ridge further reduces the pumping lift.

The conduit would be buried in a trench and backfilled so that upon completion of construction the terrain would be returned to natural contours. The route of the conduit from the pumping plant to the reservoir is along the trace defined during redesign by Reclamation in 1995. It proceeds southerly from the pumping plant, turns southwest to cross CR 211 and the Bodo creek line, passes south of hill 6966 approximately 1,200 feet south of CR 211 then turns up Bodo draw, approaches CR 211 and crosses the ridge along side CR 211. An air vent of about 12 inches diameter would stand about eight feet above ground just before the crest of the ridge. The conduit would terminate on the reservoir side of the ridge with a stilling structure from which the flow would continue down to the reservoir in a rock-lined ditch.

4.2 Conduit Profile

4.2.1 Concept of 1996 FSFES with Tunnel

The conduit described by Reclamation in The Definite Design Report, 1980, included a tunnel across Bodo ridge at elevation 6898 feet, some 80 feet below the crest of the ridge at 6878 feet. The tunnel entered the proposed 273,000 af reservoir at the level of the bottom of the 111,000 af active storage pool. The conduit at 108 inches diameter was designed for a flow of 480 cfs. Stored water could flow back down the inlet conduit, although the design criteria only required a small flow for the Animas-La Plata Water Conservancy District and the City of Durango.

4.2.2 Crest Elevation and Power Consumption

To determine the optimum elevation across Bodo Ridge the cost of construction of a tunnel at the elevation proposed in the 1996 FSFES and the cost of construction in open cut at various depths of excavation were compared with future power consumption. Normal trenching was assumed to reach a maximum depth of 16 feet. Open cut construction in excess of normal trench depth was calculated based on removing earth to provide 20 feet of pipe handling space on one side of the trench area and 30 feet of earth handling space on the other. The pipe trench area included 1/2:1 construction slopes and the handling area, 1:1 slopes. Earthwork in excess of normal trenching included open cut excavation, movement to stockpile, retrieval from stockpile, and compacting into place. Current earthwork prices were applied.

Power consumption was based on the present worth of a projected pattern of future use. The projected pattern of future use consists of initial reservoir filling over a two year period followed by a period of uniformly increasing use from 1/20th of full use the first year to full use at the end of 20 years and then full use thereafter. For comparing costs the initial filling was assumed to start the second year after the midpoint of construction expenditures. With a discount rate of 6.5 percent and a power cost inflation rate of 2.5 percent additional excavation to a trench bottom elevation of 6,945 feet resulted in the lowest cost. Table 4-1 presents costs for the complete inlet conduit including the extra cost involved with crossing Bodo ridge at the elevations investigated.

Table 4-1
Conduit Construction Cost and Present Value of Power Cost
for Various Excavation Elevations Across Bodo Ridge
Discount Rate 6.5 Percent, Cost Escalation Rate 2.5 Percent

Excavation Elevation (feet)	Construction Cost ($ million)	Present Value Power ($ million)	Total Present Value ($ million)
6962	7,200	19,300	26,500
6950	7,350	18,890	26,240
6945	7,500	18,720	26,220
6940	7,700	18,550	26,250
6930	8,130	18,200	26,330
6890 (Tunnel)	10,420	16,830	27,250

4.2.3 Conduit Diameter and Material

The design maximum flow rate for the inlet conduit is 280 cfs. The pipe would contain a pressure, or head of water, ranging from 0 to about 540 feet plus a surge pressure of about 15 percent of the head. This is in the economic pressure and diameter range of steel pipe as compared to an alternative reinforced concrete pipe. Table 4-2 presents the cost and power value investigation of alternative diameters. Surge chamber cost is included. As the pipe diameter decreases, the velocity increases and the size of the chamber required for controlling surge increases. Comparison was made at the 280 cfs flow rate because most of the reservoir refill is accomplished in the spring and early summer at the peak rate and the demand portion of the power cost is determined by this rate. A diameter of 66 inches was selected. The pipe would be coated inside and outside to protect against corrosion and equipped to accept cathodic protection if post construction measurements indicate it advisable.

Table 4-2
Conduit Diameter, Construction Cost and Present Value of Power Cost
Discount Rate 6.5 Percent, Cost Escalation Rate 2.5 Percent

Diameter (inches)	Flow Rate (cfs)	Construction Cost Conduit/Surge	Present Value Power ($ millions)	Total Present Value ($ millions)
60	280	6,600/1,610	19,580	27,790
66	280	7,500/1,340	18,720	27,560
72	280	8,400/1,130	17,270	27,800
78	280	9,525/970	17,965	28,460

4.3 Surge Provisions

Pressure variations caused by increases and decreases of flow during normal starting and stopping of pumps can be kept small by controlling the rate of movement of pump discharge valves. Pressure down surge due to power failure at the pumps, however, is a condition that requires analysis and a reliable control mechanism. Conventional means of controlling surge include air chambers, constant head tanks, and air inlet-controlled release valves.

An estimate was made of down surge following power failure to the pumps, with no surge protection on the pipeline. The result was negative or vacuum pressure extending the full length of the pipeline. This indicated the need for a large volume of air and an air chamber was selected as a reliable means of surge control. A spherical air chamber was selected for its economy of shape and with the consideration that it may also present a more pleasing appearance than an elongated tank. Investigation using Parmakian (Reclamation) charts (which assume a frictionless system) and with no reverse flow permitted through the pumps indicated a diameter of 38 feet. This diameter was selected because the air volume would limit upsurge to acceptable 15 percent and downsurge would not create negative processes.

5.0 COST ESTIMATE

5.1 Basis of Feasibility Level Cost Estimate

Estimated construction costs are based on construction quantities measured on preliminary design drawings and on unit prices selected from similar work. Major equipment items were priced based on manufacture quotations with experience-based allowances for installation. Quantities for Ridges Basin Dam were measured with an earthwork model set up over the same dam axis and ground contours used by Reclamation for the large dam considered in the 1996 FSFES, and with the dimensions of the current feasibility design for the 120,000 af dam and reservoir. Pumping plant quantities were determined from current feasibility design drawings. Unit prices based on earlier years have been updated to April 1999 using the Reclamation Construction Cost Index weighted for earth dams, pumping plants and steel pipelines.

In the estimated construction cost summary, a construction contingency amount of 20 percent considered appropriate for preliminary design level, is listed separately. Reclamation estimates have included this percentage within the itemized costs rather that listing it separately. The total field cost includes the 20 percent.

To obtain the total construction cost 30 percent is added to the total field cost based on the following estimate of work remaining to be carried out:

Item	Percent
Investigations	4
Designs and specifications	8
Construction inspection	12
Legal and Administrative	2
Environmental compliance	4
Total	**30**

5.2 Construct Cost Estimate

Table 5-1 presents a summary of the construction cost estimate. A tabular detail of quantities and estimated costs appears at the end of this section. For the total project cost, several items must be added to the estimated construction cost, including cultural resources, recreation facilities, fish and wildlife mitigation, wetlands mitigation, and interest during construction. Costs associated with the Navajo Nation Municipal Pipeline (NNMP) are discussed in Section 6.

5.3 Operating Cost

Operating cost includes operating and maintenance personnel, equipment operating and repair cost and electrical power for pumping. For future full proejct operation, pesonnel requirements were estimated to include a supervisor, records clerk, four pumping plant operators, and two maintenance workers for the Ridges Basin pumping plant, dam, reservoir, and recreation areas. In initial years, fewer personnel would be employed. Computerized supervisory control may reduce the number of pumping plant operators.

Recreation area fee collectors, boat ramp and patrol personnel are assumed to be contracted separately and covered by the fees collected. Repairs and services include annual payments made to a fund for pumping and electrical equipment repair and dam maintenance expenses that is beyond the capacity of the regular maintenance personnel. Operating costs for Ridges Basin are summarized in Table 5-2.

Table 5-1
Ridges Basin Dam, Pumping Plant and Conduit
Estimated Cost in Millions of Dollars

Item	Cost ($ million)
Ridges Basin Dam and Reservoir	**93.0**
Land Acquisition	6.7
Relocation Gas Pipelines & Electric	10.5
Relocation County Road	3.2
Access Roads	2.7
Clearing & Stock Fencing	0.8
Basin Creek Improvement	3.0
Dam Structure	57.6
Outlet Works	6.9
Initial Filling	1.6
Durango Pumping Plant	**26.8**
Land Acquisition	2.5
Access & Site Improvements	0.5
Intake Works	2.3
Plant Structure	6.2
Pumps and Motors	5.7
Manifold Piping	1.7
Auxiliary Equipment	3.0
Electrical	4.0
Surge Control	0.9
Inlet Conduit	**5.6**
Land Acquisition	0.3
Pipeline	4.8
Discharge Channel	0.5
Subtotal Field Costs, April 1999	**125.4**
Construction Contingency (20%)	24.6
Total Field Cost	**150.0**
Engineering Design, Inspection and Administrative, Legal (30%)	45.0
Total Construction Cost	**195.0**

Table 5-2
Summary of Annual Operating Costs
Ridges Basin Dam, Reservoir and Pumping Plant

	Quantity	Cost
Pumping Power		
Summer Demand	18,500 kW	381,000
Winter Demand	7,800 kW	161,000
Energy Use	68,800 Mwh	557,000
Annual PumpingCosts		**1,099,000**
Other Operating Costs		
Personnel	8 persons	320,000
Maintenance Equipment Operation		30,000
Repairs and Services		70,000
Subtotal		420,000
Total Annual Operating Cost		**1,519,000**
Power Cost $/af		**$12.18**
Project Operating $/af		**$13.68**

Notes:
Power cost based on hydrological model derived average pumping into Ridges Basin of 90,200 afy.
Rates applied: $3.43 per month per kW demand, 8.1 mils per kWh.
Project operating cost based on apportioning power cost, personnel, maintenance and repair cost to the project diversion of 111,000 afy.

6.2 Current and Projected Use

The City of Farmington has supplied treated water to NTUA since completion of the existing Farmington to Shiprock pipeline in 1969. The original 30-year contract that was recently renewed for five years with an optional additional five years provides for a maximum delivery of 3.0 million gallons per day (MGD), equal to 4.6 cfs. In 1997, NTUA drew 1,168 af from the city. Metered peak monthly flow in July 1998 was 2.64 cfs. In concept, the proposed pipeline would also receive water from the city system, although other options are under consideration. The proposed pipeline would be designed for a peak flow of 12.5 cubic feet per second (equal to 4.07 MGD), which is twice the average flow of 4,560 afy, or 6.25 cfs.

Projected water use has been based on population census, metered flows in the existing Farmington to Shiprock pipeline and production at the Shiprock water treatment plant. Projected rates of growth range from 0.69 and 1.01 percent per year (Molzen-Corbin & Associates, Navajo Tribal Utility Authority Shiprock Water Supply Study, 1993) to 2.48 percent per year (Navajo Nation Department of Water Resources Technical Memorandum, June 19, 1998). Water use rates have been affected by both the extension of service to dwellings not previously connected and leakage control and metering in the distribution systems carried out by NTUA personnel in recent years. Table 6-1 presents the current and projected water use from a 1998 base to the year 2036 when projected use would reach 4,560 afy.

Table 6-1
Farmington to Shiprock NTUA Service Area
Projected Water Use

Basis	Year	Design (MGD)	Average acre-feet/year
Current Use	1998	1.7	1,810
Molzen-Corbit	2013	3.57 to 3.88	2,400 to 2,600
Growth at 2.48%*	2036	8.07	4,560

*Highest growth rate projected. Lower rates would extend year of full capacity.

6.3 Project Elements

6.3.1 NNMP Alignment

Topography and principle water demands divide the proposed NNMP alignment into three distinct reaches. The first reach of about 13.2 miles would extend from Farmington to a high point north of Morgan Lake. It would then connect with the Farmington system, cross the San Juan River, and run along the same right-of-way as the existing pipeline south of the river. Major turnouts would supply the Upper Fruitland and San Juan Chapters. Preliminary design for this reach involves the ground elevation and supply pressure at the connection with Farmington and the ground elevation and desired system pressure at the high point in the Nenahnezad-Morgan Lake area. The existing contract with Farmington calls for a minimum pressure of 60 psi, which added to the ground elevation of 5,220 feet provides a hydraulic elevation, or grade, of 5,355 feet. The ground elevation at the high area is 5,360 feet. To provide 40 psi to the connected distribution systems would require a hydraulic grade of 5,452 feet. As such, a pumping plant would be needed between Farmington and Nenahnezad. Note that the existing pipeline is able to functions without a pumping plant because the pressure at the Farmington connection is normally higher than 60 psi. Pressures were recorded in March and May, 1999 (Dykstra 1999) and ranged from 79 to 96 psi, a hydraulic grade of 5400 to 5440 feet. A pipe diameter of 24 inches in this reach would supply turnouts along the route with adequate pressure and allow the pumping plant to be located about 1.8 miles west of Ojo Amarillo Canyon on higher ground to conserve pumping lift.

The second reach of 4.3 miles would extend from Round Knob, north of Morgan Lake to the eastern boundary of the Hogback Chapter. At the end of this reach, the pipeline would cross from the south side to the north side of the San Juan River. The diameter would be 20 inches.

The third reach of 11.2 miles would cross U.S. Highway 550 and extend to Cortez Tank at Shiprock at a top elevation of 5,170 feet. Turnouts along this reach would serve the Hogback and Sanostee Chapters and the area around Shiprock including the Cudei and Beclaibito Chapters. The storage tanks would provide local peaking flows and the pipeline diameter would be 16 inches.

The pipe diameters and the pressure or hydraulic head requirement are within the price competitive range of ductile iron; steel, cement mortar lined; steel cylinder, rod wrapped, cement mortar lined; and plastic pipe. The steel pipes would be coated with cement mortar, epoxy, or both, and may be cathodicly protected. Each of these designs, if properly installed and maintained, will give long lasting service. Final design investigations, including soil conditions and maintenance practices, would indicate if specifications should be written to invite bids on more than one type.

6.3.2 Storage Tanks

Storage tanks would be located at the high point and end of the pipeline to provide service when the pumps are not operating, to meet local peak flow needs, and to add reliability to the system for line interruptions, maintenance shut downs or fire supression. The total recommended storage capacity of 7.0 million gallons (Navajo Water Resources 1998) would provide approximately 1.7 days of reserve. Near Nenahnezad, the tank would be elevated approximately 90 feet and have a capacity of 1.5 million gallons. At Shiprock, on the hill near Cortez Tank, an additional 5.5 million gallons of ground based storage would be constructed.

6.3.3 Pumping Plant

The pumping plant would be located on a hillside about 6.9 miles from the start of the NNMP. A forebay tank would receive gravity flow through the 24-inch diameter pipeline and isolate it from pump start and shutdown surges. The pumps would lift from a hydraulic grade of about 5,275 feet to a grade of 5,455 feet, equivalent to 190 feet including plant losses. Required horsepower for the peak flow of 12.6 cfs would be 350 with a peak energy demand of 290 kW. Installation of four units of 4.2 cfs each is assumed for the cost estimate. The average flow of 6.3 cfs or 4,560 afy would use 1,250,000 kWh annually. Final design investigation would include a check-bypass alternative to the forebay reservoir. Combined with variable speed pumps, it would save energy when the Farmington delivery pressure is higher. Pump operation would be controlled by the water level in the tank in the Nenahnezad area. An air chamber on the 24-inch diameter discharge line would provide surge control.

6.4 Cost Estimate

Estimated construction costs are based on construction quantities and prices cited in the technical memoranda of the Navajo Nation Department of Water Resources (1998) and Reclamation (1999) and on unit prices selected from similar work. Earth moving along the 153,000 foot alignment would include 510,000 cubic yards (cy) of trench material, 31,900 cy of compacted pipe zone material backfill, and 412,000 cy of trench backfill. In addition there would be river and road crossings, and pumping plant and storage tank construction. Additional materials and activities would include trench excavation, pipe and appurtenances purchase and installation, and trench backfill and compaction.

Table 6-2 presents the estimated construction summary. Unit prices based on earlier years have been updated to April 1999 using the USBR Construction Cost Index. In the estimated construction cost summary, a construction contingency amount of 20 percent considered appropriate for preliminary design level, is added to obtain the total field cost. To obtain the total construction cost, 30 percent is added to the total field cost to cover additional technical investigation, engineering design, construction inspection, environmental compliance, administrative and legal costs. Table 6-2 presents the estimated construction summary.

Table 6-2
Estimated Constriction Cost Summary
Navajo Nation Municipal Pipeline

Item	Quantity	Unit Cost ($)	Amount ($)
Water line, 24 in	71,100 ft	89.00	6,327,900
Water line, 20 in	49,100 ft	70.00	3,437,000
Water line, 16 in	32,800 ft	48.00	1,574,400
Valves and Appurtenances	Lump Sum	100,000	100,000
Outlets and Transfer Connections	60 Each	800	48,000
Crossings, River	2 Each	450,000	900,000
Crossings, Roads	Lump Sum	120,000	120,000
Pumping Plant	Lump Sum	400,000	400,000
Pump Forebay	0.17 Mgal	0.35	59,500
Surge Control	Lump Sum	130,000	130,000
Storage, Ground Tank	5.5 Mgal	0.24	1,320,000
Storage, Elevated Tanks	1.5 Mgal	0.66	990,000
Subtotal Field Costs, April 1999			15,406,800
Construction Contingency (20%)			3,093,200
Total Field Cost			**18,500,000**
Engineering Design, Inspection and Administrative, Legal (30%)			5,500,000
Total Construction Cost			**24,000,000**

6.5 Operating Cost

Operating costs includes operation and maintenance personnel, equipment operation and repair cost, electrical power for pumping, and contract services. Service of regular NTUA maintenance personnel was estimated to include a part time foreman, part-time records clerk, and two full time maintenance workers. Repairs and services include annual payments made to a fund for pumping and electrical equipment repair, tank painting, and right of way maintenance expense that is beyond the capacity of the regular maintenance personnel. Operating costs for the NNMP are summarized Table 6-3.

Table 6-3
Summary of Annual Operating Costs
Navajo Nation Municipal Pipeline

	Quantity	Cost
Pumping Power		
Demand	290 kW	$ 53,500
Energy Use	1,250,000 kWh	22,500
Annual Power Cost		$ 76,000
Treated Water Cost (Purchase from City of Farmington)	2.0 MGD Average	$ 832,000
Other Operating Costs		
Personnel	2 part, 2 full time	$ 100,000
Maintenance Equipment Operation		14,000
Repairs and Services		20,000
Subtotal		130,000
Total Operating Cost		$ 1,042,000

Note:
Power rates applied: $15.40 per month per kW demand, 18 mils per kWh.
Purchase of treated water from City of Farmington at 1999 rate of $1.14 per 1000 gallons.

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ANIMAS-LA PLATA PROJECT COST ALLOCATION

The objective of cost allocations is to equitably distribute the costs of a multipurpose project among the purposes served. Since the water to be provided by the ALP Project is primarily for two purposes, municipal and industrial use and recreation, the allocation of costs performed here will be to the entities for whom the M&I water is being provided and to recreation. Law and policy require an allocation to determine reimbursable and non-reimbursable costs. The costs to be allocated include: 1) construction costs necessary to provide the identified water supplies, 2) interest on expenditures during construction, and (3) annual operation, maintenance, and replacement costs necessary to assure the project's continued operation.

Method of Allocation

Costs have been allocated in accordance with the following basic legislation:

1. Reclamation Project Act of 1939 (53 Stat. 1187) revised and amplified by the Fish and Wildlife Coordination Act of August 12, 1958 (72 Stat. 563). Provisions of these acts set up a feasible and comprehensive plan for the variable payment of construction charges on United States reclamation projects, to protect the investment of the United States in such projects, and to provide for more effective integration of fish and wildlife conservation programs with Federal water resource developments, and other purposes.

2. Public Law 485, known as the Colorado River Storage Project Authorization Act of April 11, 1956 (70 Stat. 105), and Title V of the Colorado River Basin Project Act of September 30, 1968 (Public Law 90-537). These acts authorized the Secretary of the Interior to construct, operate, and maintain the Colorado River Storage Project and participating projects. A separate fund was authorized in the Treasury of the United States to be known as the Upper Colorado River Basin Fund, which remains available until expended, for carrying out provisions of the act. Provisions of cost allocation are set for both reimbursable and nonreimbursable purposes under the Basin Fund. Provisions were also set forth under Section 8 of the act for the funding of specific recreation and fish and wildlife facilities in connection with the development of the Colorado River Storage Project and the participating projects. Title V of the Colorado River Basin Project Act, among other things, authorized the construction, operation, and maintenance of five Colorado projects, including the ALP Project, as participating projects under the Colorado River Storage Project Act.

This preliminary allocation of costs among the benefitting entities of the ALP Project follows the project plan described in the Preferred Alternative (Refined Alternative 4). The Preferred Alternative is described as a M&I water supply project with additional reservoir storage for recreation. M&I water will be provided to the following beneficiaries: the Animas-La Plata Water Conservancy District (ALPWCD), San Juan Water Commission (SJWC), Navajo Nation, Southern Ute Indian Tribe (SUIT) and Ute Mountain Ute Tribe (UMUT). The recreation storage and facility costs have been included, but a managing entity or cost sharing partner has not been identified at this point in the analysis.

The allocation methodology used is based on the amount of water supply storage provided to each benefitting entity in Ridges Basin Reservoir. Costs that could be identified specifically for the provision of water to a single entity are identified as specific costs and assigned directly to that entity. An example of a specific cost would be the cost of the Navajo Nation Municipal Pipeline which serves only the Navajo Nation. The remaining joint costs are shared among the participating entities based on the acre feet of water storage capacity each entity is provided in Ridges Basin Reservoir. An example of a joint cost would be the construction cost of Ridges Basin Dam, which serves all entities. The entities specific costs and their share of the remaining joint costs are then added together to arrive at their total allocated costs.

Costs to be Allocated

Costs to be allocated consist of construction costs of Ridges Basin Dam, Durango Pumping Plant, the Inlet Conduit, Navajo Nation Municipal Pipeline, recreation facilities and the accompanying cultural resource and mitigation costs associated with the implementation of the project. Also included are nearly $75.7 million of costs associated with previous investigations, planning, design, and environmental compliance (Sunk Costs) activities through Fiscal Year 2000. Costs for interest during construction and annual project operation, maintenance and replacement are also allocated. Table A lists the costs to be allocated in this analysis.

Interest During Construction

Reclamation requires initiation of repayment from project beneficiaries at the end of the construction period. From the time construction begins, interest during construction (IDC) accumulates on the expended construction costs. The 3.25 percent discount rate is used to compute IDC in the cost allocation but IDC is converted to the appropriate repayment rate for computing the repayment obligation. The actual repayment rate for M&I users will be determined when construction is initiated.

Separable Joint and Specific Costs

The separable cost for any purpose of a multipurpose project is the difference between the cost of the multipurpose project and the cost of the project with that purpose omitted. Thus the separable costs for each purpose include the costs of those project facilities used solely for that purpose (specific costs) plus the difference in costs of the joint-use facilities that would change in size or design with the purpose omitted (separable-joint costs). Separable costs are determined by assuming each purpose in turn as the last purpose added to the multipurpose project. The remaining joint costs are the costs remaining after the sum of the separable/specific costs for the various purposes are subtracted from the total project costs. The remaining joint costs are allocated based on respective percentage of reservoir water supply storage for each entity.

If recreation storage was omitted, Ridges Basin Reservoir could be reduced in size from 120,000 to 90,000 acre feet. The reduction in cost resulting from this elimination of recreation is estimated at $25,000,000 in construction and $910,000 for interest during construction.

Table A
Animas-La Plata Project - Costs (Apr. 1999 costs, units =$)

Item	Preferred Alternative with Recreation Pool - 120,000 af Reservoir	Preferred Alternative without Recreation Pool - 90,000 af Reservoir
Ridges Basin Dam[1]	144,600,000	119,600,000
Durango Pumping Plant[2]	41,700,000	41,700,000
Inlet Conduit	8,700,000	8,700,000
Water Acquisition Costs	40,000,000	40,000,000
Navajo Nation Municipal Pipeline	24,000,000	24,000,000
Recreation Development	12,000,000	9,000,000
Cultural Resources	7,500,000	7,500,000
Mitigation	12,100,000	12,100,000
Sunk Costs	75,700,000	75,700,000
TOTAL PROJECT COSTS TO BE ALLOCATED	366,300,000	338,300,000
Interest During Construction - (3.25% - 5 yr. period)	7,364,500	6,454,500
Annual Project OM&R		
Power	1,099,000	1,099,000
Other[3]	720,000	720,000
Total	1,819,000	1,819,000

[1]Costs include specific costs for ALPWCD of $2,000 and Colorado Ute Tribes of $7,000 to allow for future installation of water delivery systems.

[2]Costs include specific costs to ALPWCD of $930,000 for City of Durango pumping.

[3]Includes specific operation and maintenance costs for recreation facilities of $300,000.

Specific costs included the cost of $2,000 to ALPWCD and $7,000 to the two Colorado Ute tribes for hook-up facilities to allow future installation of water delivery systems. Another $930,000 in costs was assigned to ALPWCD for space in the Durango Pumping Plant to accommodate the city of Durango's pumps. An additional specific cost for the Navajo Nation Municipal Pipeline of $24,000,000 for construction and $780,000 for interest during construction has also been included.

In accordance with Public Law 93-291 (88 Stat. 174), project costs associated with the preservation of historical and archaeological data would be separately identified and shown as nonreimbursable.

The ALP Project lies within a region of significant archaeological and historical interests. Project costs of $7,500,000 and $243,750 for interest during construction have been included to inventory and preserve those sites where project facilities would be built and permanent inundation would take place. These costs have been allocated to the purpose of preservation of archaeological resources (cultural resources) Non-reimbursable costs for mitigation ($12.1 million) and cultural resources ($7.5 million) were also included along with their respective IDC costs of $393,250 and $243,750.

For this allocation, there is also the cost associated with the need to purchase lands and accompanying water rights to acquire13,000 acre feet of water for the Colorado Ute Tribes to meet the Indian Water Rights Settlement. This specific cost has been estimated at $40 million and is split equally between the UMUT and SUIT.

Annual Operation, Maintenance, and Replacement Costs

Annual operation, maintenance, and replacement costs are those expenditures necessary to assure the continued operation of the project throughout the 100-year period of analyses. The annual expenditures include such items as personnel, equipment, pumping power, supplies, replacements, administration, and other costs necessary to keep the project in efficient operating condition. For this analysis, the total annual operation, maintenance, and replacement costs are estimated at $1,819,000 including $300,000 for specific recreation facilities. An operation and maintenance estimate for the Navajo Nation Municipal Pipeline was not available at this time and therefore not included. Consistent with the analysis done at the time of authorization, CRSP power rates corresponding to the year used for construction cost estimates, were utilized for estimating pumping costs rather than the marginal replacement costs to the regional power system.

ALLOCATION OF COSTS

Table B displays the preliminary cost allocation of the Preferred Alternative.

TABLE B - ANIMAS-LA PLATA PROJECT

COST ALLOCATION (Apr. 1999 costs, including sunk costs of $75.7M, units=$)

ALLOCATION BASED ON ANNUAL RESERVOIR WATER SUPPLY DEPLETIONS - 120,000 af Reservoir

	ALPWCD	SJWC	Navajo	SUIT	UMUT	Recreation	TOTAL
WATER SUPPLY (af)	5,200	20,800	4,680	40,160	40,160	30,000	141,000
WATER DEPLETION (af)	2,600	10,400	2,340	20,080	20,080		55,500
RESERVOIR SUPPLY	4,300	4,810	1,390	40,160	40,160		90,820
PERCENT	4.73%	5.30%	1.53%	44.22%	44.22%		100.00%

Item	ALPWCD	SJWC	Navajo	SUIT	UMUT	Recreation	SUBTOTAL	Cult. Res.	Mitigation	TOTAL
Costs to be Allocated										428,012,900
Construction										366,300,000
IDC										8,144,500
OM&R (Capitalized)[4]										53,568,400
OM&R (Annual)										1,819,000
Specific Costs	962,290	0	24,780,000	20,003,614	20,003,614	46,940,800	112,690,318	7,743,750	12,493,250	132,927,318
Construction	932,000	0	24,000,000	20,003,500	20,003,500	37,000,000	101,939,000	7,500,000	12,100,000	121,539,000
IDC	30,290	0	780,000	114	114	1,202,500	2,013,018	243,750	393,250	2,650,018
OM&R (Capitalized)						8,738,300	8,738,300			8,738,300
OM&R (Annual)						300,000	300,000			300,000
Remaining Joint Costs	13,971,240	15,628,294	4,516,285	130,484,882	130,484,882	0	295,085,582			295,085,582
Construction	11,588,552	12,963,008	3,746,067	108,231,686	108,231,686	0	244,761,000			244,761,000
IDC	260,144	290,998	84,093	2,429,623	2,429,623	0	5,494,482			5,494,482
OM&R (Capitalized)	2,122,544	2,374,287	686,125	19,823,572	19,823,572	0	44,830,100			44,830,100
OM&R (Annual)	71,919	80,449	23,248	671,692	671,692	0	1,519,000			1,519,000
Total Allocations	14,933,530	15,628,294	29,296,285	150,488,496	150,488,496	46,940,800	407,775,900	7,743,750	12,493,250	428,012,900
Construction	12,520,552	12,963,008	27,746,067	128,235,186	128,235,186	37,000,000	346,700,000	7,500,000	12,100,000	366,300,000
IDC	290,434	290,998	864,093	2,429,737	2,429,737	1,202,500	7,507,500	243,750	393,250	8,144,500
OM&R (Capitalized)	2,122,544	2,374,287	686,125	19,823,572	19,823,572	8,738,300	53,568,400			53,568,400
OM&R (Annual)	71,919	80,449	23,248	671,692	671,692	300,000	1,819,000			1,819,000

[4] OM&R capitalized at 3.25 percent interest , 100 year period.

PROJECT REPAYMENT

Project beneficiaries will be responsible for paying federally funded portions of the project and the associated operation, maintenance, and replacement in accordance with Reclamation law. To determine the specific obligations of each project beneficiary and how that obligation would be contracted for will require further detailed analysis to be completed at a future time. Application of the provisions of the June 30, 1986 Cost Sharing Agreement previously entered into by the project beneficiaries could alter the total repayment obligations of the respective entities. The following is a general description of the repayment criterion for each purpose.

Municipal and Industrial

Municipal and industrial water supply construction costs and interest during construction that are Federally funded would be totally reimbursable with interest by the M&I beneficiaries at the rate in effect at the time when construction begins. Repayment of costs would be pursuant to the Water Supply Act of July 3, 1958 (72 Stat. 319). The interest during construction component of the respective beneficiaries repayment obligation would be reduced or eliminated if up-front cost sharing is applied.

Non-Reimbursable Costs

The construction costs allocated to cultural resources under Section 5 of the Colorado River Storage Act are non-reimbursable. Specific construction costs incurred for fish and wildlife mitigation pursuant to Section 8 (Public Law 84-485) are non-reimbursable, with annual OM&R costs being paid by a designated administering agency. Although Section 8 also provides for designating costs allocated to recreation as non-reimbursable, it is anticipated that such costs will be borne up-front by the recreation development entity.

OPTIONAL ALLOCATION OF COSTS

Although not included in the definition of the Preferred Alternative, two additional entities have expressed interest in receiving an allotment of project water. Their inclusion has been supported by the currently identified beneficiaries. The State of Colorado and the La Plata Conservancy District in New Mexico would share in nearly 12,000 afy of project water supply, representing a depletion of 6000 afy. To stay within the overall water depletion allowance for the Preferred Alternative, the two Colorado Ute Tribes would receive slightly less water. Table C shows an optional cost allocation which reflects the inclusion of the State of Colorado and the La Plata Conservancy District.

TABLE C - ANIMAS-LA PLATA PROJECT

OPTIONAL COST ALLOCATION - Includes State of Colorado & LCD (Apr. '99' costs, with sunk costs of $75.7M, units=$)

ALLOCATION BASED ON ANNUAL RESERVOIR WATER SUPPLY DEPLETIONS - 120,000 af Reservoir

	ALPWCD	SJWC	Navajo	SUIT	UMUT	Colorado	La Plata CD	Recreation	TOTAL
WATER SUPPLY (af)	5,200	20,800	4,680	34,160	34,160	10,440	1,560	30,000	141,000
WATER DEPLETION (af)	2,600	10,400	2,340	17,080	17,080	5,220	780		55,500
RESERVOIR SUPPLY	4,300	4,810	1,390	34,160	34,160	8,670	360		87,850
PERCENT	4.89%	5.48%	1.58%	38.88%	38.88%	9.87%	0.41%		100.00%

Item	ALPWCD	SJWC	Navajo	SUIT	UMUT	Colorado	La Plata CD	Recreation	SUBTOTAL	Cult. Res.	Mitigation	TOTAL
Costs to be Allocated												428,012,900
Construction												366,300,000
IDC												8,144,500
OM&R (Capitalized)[5]												53,568,400
OM&R (Annual)												1,819,000
Specific Costs	962,290	0	24,780,000	20,003,614	20,003,614	0	0	46,940,800	111,728,028	7,743,750	12,493,250	132,927,318
Construction	932,000	0	24,000,000	20,003,500	20,003,500	0	0	37,000,000	101,007,000	7,500,000	12,100,000	121,539,000
IDC	30,290	0	780,000	114	114	0	0	1,202,500	1,982,728	243,750	393,250	2,650,018
OM&R (Capitalized)								8,738,300	8,738,300			8,738,300
OM&R (Annual)								300,000	300,000			300,000
Remaining Joint Costs	14,443,574	16,156,649	4,668,969	114,742,441	114,742,441	29,122,277	1,209,229	0	295,085,582			295,085,582
Construction	11,980,334	13,401,257	3,872,712	95,173,998	95,173,998	24,155,696	1,003,005	0	244,761,000			244,761,000
IDC	268,939	300,836	86,936	2,136,500	2,136,500	542,256	22,516	0	5,494,482			5,494,482
OM&R (Capitalized)	2,194,302	2,454,556	709,321	17,431,943	17,431,943	4,424,325	183,709	0	44,830,100			44,830,100
OM&R (Annual)	74,351	83,169	24,034	590,655	590,655	149,912	6,225	0	1,519,000			1,519,000
Total Allocations	15,405,864	16,156,649	29,448,969	134,746,055	134,746,055	29,122,277	1,209,229	46,940,800	407,775,900	7,743,750	12,493,250	428,012,900

[5] OM&R capitalized at 3.25 percent interest , 100 year period.